Geotechnical Engineering Services
SR 305 – Poulsbo SCL to Bond Road
OL-3420
Poulsbo, Washington

September 27, 2005

For
Washington State Department of Transportation

GeoEngineers

File No. 0180-180-00

GeoEngineers

25 Years of Excellence

September 27, 2005

Washington State Department of Transportation
PO Box 47365
Olympia, Washington 98504-7365

Attention: William S. Hegge, PE

We are pleased to submit our report titled "Geotechnical Engineering Services, SR 305 – Poulsbo SCL to Bond Road, OL-3420, Poulsbo, Washington." Our services were completed as Task AD under existing Agreement No. Y-8473 for On-Call Geotechnical Engineering Services. We provided preliminary results to you in project meetings; recommendations in this report are consistent with those previously given.

We appreciate the opportunity to provide geotechnical engineering services on this interesting project, and we are available to meet with the project team to discuss the information presented in this report. Please call if you have any questions, or if you require additional information.

Respectfully submitted,

GeoEngineers, Inc.

Galan W. McShelly, Principal

for Daniel J. Campbell, PE
Principal

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Prepared for:

Washington State Department of Transportation
PO Box 47365
Olympia, WA 98504-7365

Attention: William S. Hegge, PE

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OL-3420
POULSBO, WASHINGTON
FOR
WASHINGTON STATE DEPARTMENT OF TRANSPORTATION

1.0 INTRODUCTION

1.1 GENERAL

This report presents the results our geotechnical engineering services for specific aspects of the SR 305 – Poulsbo SCL to Bond Road, OL-3420 project. Our involvement included providing geotechnical recommendations for nine retaining walls and three, 3-sided, open-bottom culverts. The project is located in Poulsbo, Washington as shown on the Vicinity Map, Figure 1.

The purpose of our services was to assess the subsurface soil and groundwater conditions as a basis for providing detailed geotechnical recommendations for use in developing plans, specifications and estimates for the subject project. The work was completed as Task AD under existing Agreement Number Y-8473 for On-Call Geotechnical Engineering Services between the Washington State Department of Transportation (WSDOT) and GeoEngineers, Inc. (GEI).

1.2 PROJECT UNDERSTANDING

Our understanding of the project is based on discussions with Bryan Dias and Bill Hegge of the WSDOT Headquarters Geotechnical Division and copies of project documents that were provided to us. These documents include a request for proposal letter dated May 31, 2005, preliminary plans, a previous geotechnical report for the project corridor prepared by HWA GeoSciences (1999), and a preliminary exploration plan for supplemental project borings prepared by WSDOT. Our services are directed toward providing geotechnical input for three, three-sided, open-bottom culverts and nine retaining walls, and not the entire project corridor. In essence, we are supplementing the existing geotechnical report for the project.

We understand the improvements for SR-305 project will ultimately include the addition of a northbound and southbound lane along SR-305 from the south Poulsbo City limit to Bond Road (SR 307). The project will also include the addition of turning lanes at seven intersections, sidewalks and bicycle lanes. A 1,200-foot section of South Fork Dogfish Creek will be realigned along the east side of SR 305. Open bottom culverts will replace existing culverts at three locations to provide creek improvements for aquatic life.

This report focuses on the retaining wall and culvert improvements. Our understanding of the proposed culverts and walls is summarized in Table 1 below.

Table 1. Culvert and Retaining Wall Information

Culverts				
Structure Number	Location, Stationing	Length (ft)	Span (ft)	Height (ft)
1	SR 305, A723+45	107	10	12½[1]
2	Bond Road, BR 54+50	80	16	11[1]
3	Bond Road, BR 43+34	75	12	13[1]

Retaining Walls				
Wall Number	Location, Stationing	Length (ft)	Cut/Fill	Maximum Height (ft)
5	Lincoln Road, A 723+44.49 to LD 48+42.26 (Rt.)	165½	Fill	16[1]
8	SR 305, A 749+50 (Rt.) to 761+00 (Rt.)	1,150	Fill	16½
10	SR 305, A 762+66.48 (Rt.) to 776+12 (Rt.)	1,345½	Fill	13½
11	SR 307, BR 54+07 (Lt.) to 55+00 (Lt.)	93	Fill	12½[1]
12	SR 307, BR 54+07 (Rt.) to 55+12 (Rt.)	93	Fill	15[1]
13	Lincoln Road, B 50+88.42 (Rt.) to LD 48+18.59 (Lt.)	71½	Fill	6
14	Bond Road, BR 43+36.90 (Rt.) to 43+71.40 (Rt.)	46½	Fill	10[1]
15	Bond Road, BR 43+07.70 (Lt.) to 43+38.20 (Lt.)	67½	Fill	11½[1]
16	Bond Road, BR 56+13.58 (Rt.) to 60+00 (Rt.)	488	Fill	9

Note:
[1] Heights extend to bottom of adjacent culvert foundations.

The proposed retaining wall and culvert improvement locations for the project are presented on the Site Plan, Figures 2A to 2E.

1.3 Scope Of Services

The purpose of our geotechnical services is to characterize the existing subsurface soil and groundwater conditions to provide design recommendations for the retaining walls and culvert structures. Our services for this project were completed in general accordance with our proposal dated June 3, 2005. Written authorization to proceed with our services was provided by WSDOT on June 10, 2005 under Task Assignment AD of Consultant Agreement Y-8473. Our scope of services includes providing geotechnical recommendations for:

Culverts

- Feasible foundation types and recommend most suitable;
- Estimates of settlement;
- Seismic design parameters and evaluation of liquefaction potential;
- Allowable foundation capacity for an LFD design approach;
- Foundation capacity presented in graphs as a function of foundation width and resistance factors in LRFD format;
- Provide recommendations for wall backfill and design lateral earth pressures; and
- Comment on construction considerations.

Retaining Walls

- Feasible wall types and recommend most suitable;
- Bearing capacity, settlement, and external stability. For non-standard, non-proprietary walls internal stability is also addressed;
- Seismic design parameters; and
- Construction considerations.

1.4 Previous Studies

A previous study was completed by HWA Geosciences in 1999 for this project. This study, along with the exploration logs, was made available to GeoEngineers. The data from the previous study was used to aid interpretation of subsurface conditions at the walls and culverts. Where appropriate, simplified logs from this previous study are included on plan, profile, and elevation views of the proposed retaining walls and culverts. Please refer to the original study for copies of the actual boring logs.

2.0 SITE CONDITIONS

2.1 SURFACE CONDITIONS

The portion of the project that includes the nine retaining walls and the three culverts is located along SR 305 beginning at the intersection with NE Lincoln Road and extending north to Bond Road. The project also includes several walls and two culverts located east and west of the intersection of SR 305 and Bond Road. The improvements along Bond Road are between 1st Avenue NE and Big Valley Road NE, which are situated west and east of the intersection of SR 305 and Bond Road, respectively.

The topography along the alignment of SR 305 is relatively flat with a gradual decrease in elevation from south to north. The topography in the vicinity of the intersection of SR 305 and Bond Road is relatively level. The topography slopes up gently to moderately toward the east along Bond Road. Observations of the roadway alignment indicate fills were used to construct SR 305 and a combination of cuts and fills were used to construct the Bond Road alignment. Gently to steeply sloped fill embankments are located along the east and west sides of SR 305. Steep slopes are present in the vicinity of some of the existing culverts along Bond Road that will be replaced with Structure Nos. 2 and 3.

The land along either side of SR 305 is typically occupied by commercial businesses. The improvements are typically set back from the road way a sufficient distance as to not impact the plans to widen the highway. The land along the south side of Bond Road is typically occupied by commercial businesses including a nursery that is located east of NE Bernt Road along the Bond Road alignment.

Vegetation along the alignment consists of deciduous and coniferous trees, blackberries, grasses and other shrubs. The undergrowth is quite heavy along portions of SR 305. Ornamental landscaping is present along portions of the SR 305 and Bond Road alignments.

Surface water features include South Fork Dogfish Creek which extends along the east side of SR 305. This creek is located near the proposed alignments of Walls 8 and 10. The creek also is present in the vicinity of the intersection of SR 305 and NE Lincoln Road and passes through an existing structure below SR 305 that will be replaced with Structure No. 1. This portion of the creek will pass through both Walls 5 and 13. The creek also crosses below Bond Road through culverts that will be replaced by 3-sided structures Nos. 2 and 3. The head walls for each of these structures (Walls 11, 12, 14 and 15) will be constructed perpendicular to the creek alignment at either end of the 3-sided structures. Additional water features include a wetland that exists near the north end of Wall 8 at the bottom of the roadway fill.

2.2 REGIONAL GEOLOGY

The geology of the Puget Sound region includes a thick sequence of overconsolidated glacial and unconsolidated non-glacial soils overlying bedrock. Glacial deposits were formed by ice sheets originating in the mountains of British Columbia. The most recent glacial advance was the Fraser Glaciation, which included the Vashon Stade, during which the Puget Lobe of the continental ice sheet advanced and retreated through the Puget Sound Basin. The Vashon Stade occurred approximately 14,000 years before the present time and is the source of the glacial deposits in the project area. Landforms within the project area are primarily the result of glaciation, erosion, sedimentation, stream deposition, and modification by road building activities.

Published geologic information for the project vicinity includes a U.S. Geological Survey Map titled "Geology and Groundwater Resources of Kitsap County, Washington" (Sceva, 1957). Four geologic units are identified along the project alignment in addition to man-placed fill consisting of variable quantities of sand, gravel and/or silt. The geologic units include: alluvial deposits, glacial till, advance outwash, and transitional deposits. The engineering characteristics of these soils are discussed in **Section 3.2** of this report.

Published geologic hazard areas for the project vicinity include erosion hazards, seismic hazards, landslide hazards, and steep slope hazards. The data for these hazards was provided by Kitsap County (2005). Portions of the project corridor lie in or adjacent to areas designated as Areas of Geologic Concern, but the specific hazard types and locations are not specified. Based on our field reconnaissance, the primary geologic hazards at the proposed wall locations include steep slopes and erosion hazards. During our reconnaissance we observed that the steep slopes appear to be stable at this time and no significant erosion is taking place.

2.3 REGIONAL SEISMICITY

2.3.1 Introduction

The Puget Sound area is located at the convergent continental boundary known as the Cascadia Subduction Zone (CSZ). The CSZ is the zone where the westward advancing North American plate is overriding the subducting Juan de Fuca plate. The CSZ extends from mid-Vancouver Island to Northern California. The interaction of these two plates results in three potential seismic source zones. These three seismic source zones are: (1) the shallow crustal source zone, (2) the Benioff source zone, and (3) the CSZ interplate source zone.

2.3.2 Shallow Crustal Earthquakes

The shallow crustal source zone is used to characterize shallow crustal earthquake activity within the North American plate. Shallow crustal earthquakes typically occur at depths ranging from 3 to 18 miles. The shallow crustal source zone is characterized as being capable of generating earthquakes up to about magnitude 7.5. Large shallow crustal earthquakes are typically followed by a sequence of aftershocks. Shallow crustal faults with known or suspected displacements within the general project area include the Seattle Fault zone and the Southern Whidbey Island Fault.

The Seattle Fault zone is located approximately 10 miles to the southeast of the project. The Seattle Fault zone is a 2½- to 4-mile-wide, east-west trending zone of three or more south dipping reverse faults (Johnson et al., 1999). The Seattle Fault ruptured about 1,100 years ago and caused broad uplift and subsidence on either side of the fault. The rate of recurrence of large earthquakes on the Seattle Fault is thought to be on the order of thousands of years.

The Southern Whidbey Island Fault is located approximately 19 miles to the northeast of the project. The Southern Whidbey Island Fault is a northwest trending reverse- and strip-slip fault structure extending from the southern end of Whidbey Island to the Strait of Juan de Fuca. The most recent major earthquake and fault displacement is estimated to have occurred about 3,000 years ago (Kelsey et. al., 2003).

2.3.3 Benioff Source Zone Earthquakes

Benioff source zone earthquakes are also referred to as intraplate, intraslab, or deep subcrustal earthquakes. Benioff zone earthquakes occur within the subducting Juan de Fuca plate between depths of 20 and 40 miles and typically have no large aftershocks. Extensive faulting results as the Juan de Fuca plate is forced below the North American plate and into the upper mantle.

The Olympia 1949 (M = 7.1), the Seattle 1965 (M = 6.5), and the Nisqually 2001 (M = 6.8) earthquakes are considered to be Benioff zone earthquakes. The Benioff zone is characterized as being capable of generating earthquakes up to magnitude 7.5. The recurrence interval for large earthquakes originating from the Benioff source zone is believed to be shorter than for the shallow crustal and CSZ source zones—damaging Benioff zone earthquakes in Western Washington occur every 30 years or so. The deep focal depth of these earthquakes tends to dampen the shaking intensity when compared to shallow crustal earthquakes of similar magnitudes.

2.3.4 Subduction Zone Earthquakes

The Cascadia Subduction Zone is an approximately 650-mile long thrust fault that extends along the Pacific Coast from mid-Vancouver Island to Northern California. CSZ interplate earthquakes result from rupture of all or a portion of the convergent boundary between the subducting Juan de Fuca plate and the overriding North American plate. The fault surfaces approximately 50 to 75 miles off the Washington coast. The width of the seismogenic portion of the CSZ interplate fault varies along its length. As the fault becomes deeper, materials being faulted become ductile and the fault is unable to store mechanical stresses.

The CSZ is considered as being capable of generating earthquakes of magnitude 8 to magnitude 9. No earthquakes on the CSZ have been instrumentally recorded; however, through the geologic record and historical records of tsunamis in Japan, it is believed that the most recent CSZ event occurred in the year 1700 (Atwater, Brian F, 1996 and Satake, K, et. al, 1996). Recurrence intervals for CSZ interplate earthquakes are thought to be on the order of 400 to 600 years. Paleogeologic evidence suggests five to seven interplate earthquakes may have been generated along the CSZ over the last 3,500 years at irregular intervals.

3.0 SUBSURFACE CONDITIONS

3.1 GENERAL

Subsurface conditions along the project corridor were evaluated by reviewing previous geotechnical explorations and laboratory data and by completing seven supplemental borings and four hand explorations.

The borings (TH-40-05 through TH-46-05) were drilled by WSDOT using cased mud rotary drilling techniques on June 9-12, 2005 and August 1-2, 2005. The borings extended to depths of 20½ to 51½ feet below the existing ground surface. The hand explorations (HH-1 through HH-4) were performed by GeoEngineers using hand tools on June 12-13, 2005 and August 2, 2005. The hand explorations extended to depths of 1½ to 5 feet.

The approximate locations of the supplemental explorations completed for this project as well as the pertinent explorations from the previous study are presented on the Site Plan, Figures 2A through 2E. Details of the field exploration program and logs of the explorations completed for this study are presented in Appendix A. A few field vane shear tests were performed in conjunction with boring TH-46-05. The results of the field vane shear test are also presented in Appendix A.

Soil samples were collected during the exploration program and taken to GeoEngineers' laboratory for further evaluation. Selected samples were tested for the determination of moisture content, fines content, grain size distribution, and Atterberg limits (plasticity characteristics). Consolidation and triaxial shear strength tests were also completed on selected samples. A description of the laboratory testing and the test results are presented in Appendix B.

3.2 Soil Units and Engineering Characteristics

3.2.1 General

Subsurface conditions observed in the supplemental explorations performed at the site were generally consistent with geologic maps and previous explorations. The subsurface soils along the project corridor generally consist of five soil units: fill, alluvial deposits, glacial till, advance outwash, and transitional deposits. These soil units and their typical engineering characteristics are presented below, beginning with the most recently deposited. It is important to note that the engineering properties described are general in nature. The boring logs should be reviewed to assess subsurface conditions and engineering characteristics of the soils at specific locations.

3.2.2 Fill

Fill was encountered in many of the explorations near proposed wall locations. The fill generally appears to be associated with the construction of existing roads. The fill material ranges in density from very loose to medium dense and typically consists of sand, silt and gravel. The fill observed in the explorations varies in thickness from 2 to 13 feet. In general, because of the variability in density of this material, the fill may not provide adequate foundation support for walls or culverts unless measures are taken to improve the density.

The existing fill soils generally meet the criteria for "Common Borrow" as described in Section 9-03.14(3) of the WSDOT (2004) "Standard Specifications." However, portions of the fill have relatively high fines content (material passing the U.S. No. 200 sieve) and will therefore be moisture sensitive. These soils may become muddy and unstable when exposed to moisture. It will also be difficult to operate equipment on or adequately compact these soils during wet weather conditions because of the high fines content.

3.2.3 Alluvial Deposits

Alluvial deposits were encountered in the vicinity of proposed Walls 5, 8, and 10 and Culvert Structure No. 1. Alluvial deposits originate from Holocene period (post-glacial) river and stream flows and have not been glacially consolidated. The alluvial deposits observed typically consist of very soft to very stiff/dense, stratified deposits of silt and clay with layers of sand containing variable silt content. Organics were encountered within the alluvial deposits in the vicinity of Walls 8 and 10. The alluvial deposits observed in the explorations vary in thickness from 1½ to 22 feet. Laboratory tests indicate that the silt and clay layers are compressible (see Appendix B for test results). Our analyses indicate that granular portions of the alluvial deposits located below the groundwater table may be susceptible to liquefaction. Depending on the consistency of the alluvial material at the foundation elevation, this material may not provide adequate foundation support for walls or culverts and some remedial excavation may be necessary. In addition, alluvial soils will be subject to consolidation which may result in wall settlement and portions of these deposits are susceptible to liquefaction.

Alluvial deposits with relatively high fines content will be moisture sensitive and will become muddy and unstable when the amount of moisture in the soil rises above the optimum moisture content. Provided the material is granular and organic material is separated from these soils or is present in minor amounts, these soils generally meet the requirements for "Common Borrow." In general, the fine grained portions of the alluvial deposits (silt and clay) do not meet the criteria for "Common Borrow".

3.2.4 Glacial Till

Glacial till was encountered in one boring in the vicinity of proposed walls 11 and 12 (TH-41-05). Glacial till typically consists of a very dense, nonsorted mixture of silt, sand, gravel, and cobbles that was deposited during glaciation and consolidated by the weight of the ice. Boulders are often encountered in glacial till. The glacial till encountered in TH-41-05 was approximately 6 feet thick. This material generally should provide excellent foundation support for walls or culverts.

Glacial till typically contains a significant percentage of fines (silt and clay) and is moisture sensitive. When the moisture content is more than a few percent above the optimum moisture content, glacial till soils become muddy and unstable and operation of equipment on these soils can be difficult. Glacial till soils typically meet the criteria for "Common Borrow."

3.2.5 Advance Outwash

Advance outwash was encountered in the vicinity of Walls 5, 11, 12, 13, 14, 15, and 16. Advance outwash typically consists of medium dense to very dense, stratified sand with gravel and occasional cobbles. Small boulders are often encountered in advance outwash. Most of the explorations were terminated in the advance outwash but the deeper explorations show the advance outwash is generally underlain by transitional beds. This material generally should provide excellent foundation support for walls or culverts.

Advance outwash deposits often contain relatively low fines content. Locally, the advance outwash can be silty and contain layers of fine-grained sands and silts. Advance outwash soils are typically less moisture sensitive than glacial till soils. Advance outwash sand and gravel often meet the gradation requirements for "Gravel Borrow" and "Select Borrow" as described in Sections 9-03.14(1) and 9-03.14(2), respectively, of the WSDOT (2004) "Standard Specifications." Locally silty lenses typically meet the criteria for "Common Borrow."

3.2.6 Transitional Deposits

Transitional deposits were encountered in borings TH-42-05 and TH-44-05 in the vicinities of Structure No. 3 (Walls 14 and 15) and Wall 10, respectively. Transitional deposits typically consist of thick sections of hard clay and silt. The transitional deposits in borings TH-42-05 and TH-44-05 extend to the depths explored. Transitional deposits typically have a significant probability of slope instability if present in sloping ground areas. High moisture contents, plasticity and jointing are associated with these deposits. In addition, zones of seepage can occur above these deposits in sloping situations because of the relatively low permeability of the transitional deposits. However, this material was found at depth along the corridor, and our slope stability analyses indicate adequate factors of safety for Walls 10, 14 and 15.

Transitional deposits typically contain mostly fines (silt and clay) and are moisture sensitive. When the moisture content is more than a few percent above the optimum moisture content, these soils become muddy and unstable and operation of equipment on these soils can be difficult. Transitional deposits will not meet the criteria for "Common Borrow" and are not recommended for re-use as structural fill.

3.3 Groundwater Conditions

Variable groundwater conditions were observed in the existing and the supplemental borings completed along the project corridor. The depth to groundwater measured during drilling and in the piezometers of the supplemental borings was as shallow as 1 feet below the ground surface and ranged up to about 15 feet below the ground surface. We anticipate the groundwater level along the project corridor will fluctuate as a function of season, precipitation and other factors.

:r elevations observed during drilling and measured in piezometers.

le 2. Groundwater Information

		pproximate Elevation of ındwater Observed During Drilling (ft)[1]	Groundwater Measured in Piezometer	
			Date	Approximate Elevation (ft)[2]
[illegible]	42	39	09/09/05	36½
TH-41-05	32	28	09/09/05	23
TH-42-05	15	12	09/09/05	13
TH-43-05	33	27		
TH-44-05	36	31	09/09/05	35
TH-45-05	40	39	09/09/05	38
TH-46-05	46	31		
HH-1	23	None		
HH-2	17	15		
HH-3	100	99		
HH-4	35	None		

Notes:

[1] The groundwater levels observed during drilling were measured in the drill casing prior to removal or shortly after well installation. This water level may not truly represent actual groundwater elevation.

[2] The water level measured in a piezometer is representative of a static groundwater condition.

3.4 Subsurface Conditions Along Wall Alignments

3.4.1 General

The locations of the walls, culverts, supplemental and pertinent existing explorations are shown on the Site Plan, Figures 2A through 2E. A plan view of each wall showing the exploration locations along with elevation and section views are also included in this report. The elevation and section views show our interpretation of the subsurface soil and groundwater conditions for each retaining wall. These plan, elevation and cross section views are presented in Figures 3 through 19.

3.4.2 Wall 5

Wall 5 is located at the southwest corner of the intersection of NE Lincoln Road and SR 305. Figure 3 shows a plan and elevation view of the wall. Figure 4 presents Cross Section A-A' through the wall. The subsurface conditions generally consist of embankment fill over advance outwash. There is a relatively thin layer of alluvium, approximately 5 feet thick, separating the fill and advance outwash soils along the southern portion of the wall that will run along SR 305. The fill and alluvium typically consist of very loose to medium dense silty sand. The advance outwash consists of dense to very dense sand and gravel.

There are several abandoned culverts running beneath the roadway embankments as well as numerous utilities including storm drain and gas. Overhead power, telephone, television and fiber optic are also present along the south side of NE Lincoln Road.

3.4.3 Wall 8

Wall 8 is located on the east side of SR 305 just south of Forest Rock Lane NE. Figure 5 shows a plan and elevation view of the wall, and Figures 6 and 7 present Cross Sections B-B' and C-C' through the wall. The subsurface conditions generally consist of embankment fill over alluvium over transitional deposits. The fill and alluvium along the proposed face of wall are up to about 20 feet thick on the south end of the wall (cross section C-C') and thin to approximately 5 to 10 feet on the north end of the wall (cross section B-B'). The fill typically consists of loose to medium dense silty sand. The alluvium typically consists of soft to stiff silt and clay. The underlying transitional deposits generally consist of hard silt and clay with medium dense to very dense sand.

Portions of South Fork Dogfish Creek will need to be relocated to accommodate the construction of the wall as the creek alignment currently coincides with the face of the wall. In addition, an existing wetland area is located near the north end of the wall. The presence of these surface water features suggests the likelihood of soft wet soils near the ground surface and relatively high groundwater conditions.

3.4.4 Wall 10

Wall 10 is located on the east side of SR 305 just north of Forest Rock Lane NE. Figure 8 shows a plan and elevation view of the wall, and Figure 9 presents Cross Section D-D' through the wall. The subsurface conditions generally consist of embankment fill over alluvium underlain by transitional deposits. The fill and alluvium along the proposed face of wall are up to 21 feet thick; although these layer are only a few feet thick at the north end of the wall. The fill typically consists of loose to medium dense silty sand. The alluvium typically consists of very soft to stiff silt and clay. The transitional beds consist of interbedded layers of medium dense to dense sand and stiff to hard silt and clay.

Similar to Wall 8, portions of South Fork Dogfish Creek will need to be relocated to accommodate the construction of the wall as the creek alignment currently coincides with the face of the wall. The presence of surface water features suggests the likelihood of soft wet soils near the ground surface and relatively high groundwater conditions.

3.4.5 Walls 11 and 12

Walls 11 and 12 are located on the north and south sides of Bond Road, respectively, just east of the intersection with NE Bernt Road. These walls form the head wall at the ends of Culvert Structure No. 2. Figure 10 shows a plan and elevation view of the walls; Figure 11 presents Cross Section E-E' through Wall 11, and Figure 12 present Cross Section F-F' through Wall 12. The subsurface conditions generally consist of a thin layer of embankment fill over glacial till and advance outwash.

At exploration TH-41-05 the fill is approximately 3 feet thick and consists of loose silty sand. The fill is underlain by dense to very dense glacial till that is about 7 feet thick. The boring was terminated in the very dense advance outwash sand located below the glacial till.

3.4.6 Wall 13

Wall 13 is located at the northeast corner of the intersection of NE Lincoln Road and 8th Avenue NE. Figure 13 shows a plan and elevation view of the wall, and Figure 14 presents Cross Section G-G' through the wall. The subsurface conditions generally consist of embankment fill over advance outwash. The fill typically consists of loose to medium dense silty sand. Based on the explorations, dense to very dense advance outwash sand and gravel was observed about 3 to 5 feet below the existing grade elevation.

South Fork Dogfish Creek crosses below NE Lincoln Road through an existing culvert at the location of Wall 13. The presence of the creek suggests the likelihood of soft wet soils near the ground surface and relatively high groundwater conditions.

3.4.7 Walls 14 and 15

Walls 14 and 15 are located on the north and south sides of Bond Road, respectively, just east of the intersection with 1st Avenue NE. These walls form the headwalls at the ends of proposed Culvert Structure No. 3. Figure 15 shows a plan and elevation view of the walls; Figure 16 presents Cross Section H-H' through Wall 14, and Figure 17 present Cross Section I-I' through Wall 15. The subsurface conditions generally consist of embankment fill over advance outwash underlain by transitional deposits.

At exploration TH-42-05, which was completed near the proposed alignment of Wall 15, the fill is approximately 6 feet thick, and the advance outwash is about 5½ feet thick. The fill typically consists of loose to medium dense silty sand. The advance outwash generally consists of very dense silty sand, and the transitional deposits consist of hard clay. The boring was terminated in the transitional deposits.

South Fork Dogfish Creek crosses below Bond Road through an existing culvert at the location of these walls. The presence of creek suggests the likelihood of soft wet soils near the ground surface and relatively high groundwater conditions.

3.4.8 Wall 16

Wall 16 is located on the south side of Bond Road just east of NE Bernt Road. Figure 18 shows a plan and elevation view of the wall, and Figure 19 presents Cross Section J-J' through the wall. The subsurface conditions generally consist of embankment fill over advance outwash. The fill typically consists of loose to medium dense silty sand and ranges up to about 5 or 6 feet based on boring TH-40-05. The advance outwash soils below the fill typically consist of dense to very dense sand.

4.0 GEOTECHNICAL CONCLUSIONS AND RECOMMENDATIONS

4.1 EARTHQUAKE ENGINEERING

4.1.1 Design Parameters

The seismic design of the walls and culverts can be completed using the design criteria presented in the WSDOT Geotechnical Design Manual (GDM). The design manual references the 2002 USGS National Seismic Hazards Mapping project for determining a peak ground (bedrock) acceleration coefficient for design. A peak bedrock acceleration coefficient of 0.32 should be used for determination of AASHTO generalized response spectra based on the National Seismic Hazard Maps. A peak ground acceleration of 0.38 should be used for evaluating ground response (liquefaction, slope stability, etc.); this is the bedrock acceleration value provided in the 2002 USGS mapping that has been factored to include amplification effects associated with soil (Stewart et. al., 2003). The acceleration coefficients are based on the expected ground motion at the project site that has a 10 percent probability of exceedance in a 50-year period (475-year return period).

4.1.2 Liquefaction Potential

Liquefaction refers to a condition where vibration or shaking, usually from earthquake forces, results in the development of excess pore water pressures in saturated soils causing loss of soil strength. Ground settlement, lateral spreading and/or sand boils may result from liquefaction. Structures supported on liquefied soils could suffer foundation settlement or lateral movement that could be severely damaging to the structures. In general, soils susceptible to liquefaction include loose to medium dense saturated cohesionless soils, but can occur in soils with grain sizes varying from silt to gravel.

The evaluation of liquefaction potential is complex and is dependent on numerous site parameters including soil grain size, soil density, age of the soil deposit, site geometry, static stresses and design accelerations. Typically the liquefaction potential of a site is evaluated using the Simplified Procedure (Youd et al. 2001). The Simplified Procedure is based on comparing the cyclic resistance ratio (CRR) of a soil layer (the cyclic shear stress required to cause liquefaction) to the cyclic stress ratio (CSR) induced by an earthquake. The factor of safety against liquefaction is determined by dividing the CRR by the CSR. A detailed description of the method is available in Section 6.5.2.1 of the GDM. In accordance with the GDM, liquefaction hazards, including settlement and related effects, were evaluated when the factor of safety against liquefaction was calculated as less than 1.2 using the simplified procedure.

Table 3 below summarizes our liquefaction assessment, which shows potential for liquefaction at Walls 8 and 10. The results of our analyses indicate that the potential for liquefaction at the other walls and 3-sided culvert structures was not significant as these walls and culverts would be likely be founded on dense to very dense glacially consolidated soils.

Table 3. Liquefaction Evaluation Results

Wall	Estimated Thickness of Potentially Liquefiable Soils (ft)	Estimated Elevations of Potential Liquefaction (ft)	Estimated Liquefaction-Induced Settlement (in)
Wall 8	5 to 8	30 to 27	1 to 2
Wall 10	5 to 10	30 to 25	1 to 2

The settlement values presented in Table 3 represent the anticipated total settlement as a result of liquefaction. Because of the variability in the soils and the fact that the amount of soil that liquefies within the liquefiable layers will likely not be uniform, the differential settlement along 25 feet of wall is anticipated to be approximately equal to the total settlement.

The post-liquefaction slope stability of the proposed walls was evaluated using the computer program SlopeW version 5.20 (GEO Slope International, Ltd, 2004). The residual undrained shear strength of the liquefiable soils was used in the post-liquefaction evaluation. The values of the residual undrained shear strength were estimated consistent with Section 6.2.2 of the GDM and are based on Seed and Harder, 1990. We used values from the lower half of the published range for our analysis. The actual residual undrained shear strength values (expressed as cohesion) used for each wall, along with the other soil input parameters, are presented on the individual slope stability results figures in Appendix C.

The results of our analysis indicate factors of safety for the post liquefaction condition are greater than 1.3. Based on the results of our analyses, we do not anticipate global slope failures as a result of liquefaction below proposed Walls 8 and 10.

4.1.3 Ground Rupture

Because of the thickness of the non-glacially and glacially consolidated soils below the site and the estimated distance to the closest known fault (approximately 10 miles), the potential for surface fault rupture at the site is considered to be low.

4.2 RETAINING WALLS

4.2.1 General

Based on the results of our site reconnaissance, explorations, laboratory testing, and engineering analyses it is our opinion that the construction of the proposed retaining walls is feasible from a geotechnical standpoint provided the recommendations in this report are incorporated into the design and construction. Several retaining wall options are suitable for the walls. With the exception of Wall 13, the walls have maximum exposed heights that typically exceed 10 feet and therefore prefabricated modular walls such as gabion walls or ecology block walls are likely not suitable, particularly considering the wall heights will generally be about 2 feet higher than the exposed height when the wall embedment is included. Mechanically Stabilized Earth (MSE) walls and standard plan concrete cantilever walls are suitable wall types that have been pre-approved by WSDOT and will work well for most of these walls. Because of the close proximity of underground utilities and a desire not to relocate them, we understand that WSDOT desires to construct a cantilever soldier pile wall for Wall 5.

We anticipate that MSE walls will be used extensively on this project. MSE walls are suitable for supporting fills and for the wall heights required for this project. They are often a cost effective option and are more settlement tolerant than concrete cantilever walls. This will be particularly advantageous for the construction of Walls 8 and 10 which will be constructed over fill and alluvial deposits that are expected to settle under the loads of the proposed walls.

During conversations with WSDOT representatives throughout the project, the wall types for Walls 5, 8, 10, and 16 have been identified and are presented in Table 4 below. Feasible alternative wall types are provided for the remaining walls. Design information and construction considerations for the retaining walls are also presented in Table 4 below.

Table 4. Retaining Wall Design Information

Wall	Length (ft)	Maximum Exposed Height (ft)	Feasible Wall Types	Design/Construction Considerations
5	165½	16[1]	Cantilever Soldier Pile	• Steep slopes limit access for installing piles. • Overhead utility conflicts. • Caving soils may require use of cased holes to install piles. • Construction will be in creek. • Wall will need to incorporate Structure No. 1 and 8-ft CMP culverts into wall design. This will result in wide pile spacing adjacent to culvert openings. • Staging for construction of Structure No. 1 below SR 305 and Wall 13 will need to be coordinated.
8	1150	16½	MSE	• Temporary cuts to install reinforcing may impact roadway. • Remove and replace up to 2 feet of soft soils at foundation subgrade elevation to provide adequate bearing for wall. The backfill may need to consist of crushed rock if construction takes place in presence of GWT. A separator fabric will likely be required. • Construction will be in creek and wetland • Construction must accommodate existing culverts • Creek must be relocated to front of wall • Relocate light poles • Analysis indicates 3 to 6 inches of wall settlement possible
10	1345½	13½	MSE	• Temporary cuts to install reinforcing may impact roadway. • Remove and replace up to 2 feet of soft soils at foundation subgrade elevation to provide adequate bearing for wall. The backfill may need to consist of crushed rock if construction takes place in presence of GWT. • Construction will be in creek. • Construction must accommodate existing culverts. • Creek must be relocated to front of wall. • Analysis indicates 2 to 4 inches of wall settlement possible.
11	93	12½[1]	Concrete Cantilever or MSE Wall	• Temporary cut slopes for the wall will impact existing roadway, however wall will likely be constructed when culvert Structure No. 2 is built and road is partially closed. • Temporary shoring likely required for construction of Structure No. 2. • Wall should extend to base of culvert structure and have sufficient embedment below anticipated scour depth. • Localized removal and replacement of soft alluvial soils with structural fill. • Construction will be in creek. • Staged construction with Wall 12.

Wall	Length (ft)	Maximum Exposed Height (ft)	Feasible Wall Types	Design/Construction Considerations
12	93	15[1]	Concrete Cantilever or MSE Wall	• Similar issues as presented above for Wall 11.
13	71½	6	Concrete Cantilever or MSE Wall	• Temporary cuts to install reinforcing or footing may impact roadway. • Localized removal of soft soils. The backfill may need to consist of crushed rock if construction takes place in presence of GWT. • Construction will be in creek. • Construction may occur with installation of 8-ft CMP culvert. • Coordinate construction with Wall 5.
14	46½	10[1]	Concrete Cantilever or MSE Wall	• Partial road closure required to replace existing culvert with Structure No. 3. • Construction must accommodate orphan culverts. • Remove existing rockery during construction. • Temporary shoring likely required for installation of Structure No.3. • Wall should extend to base of culvert structure and have sufficient embedment below anticipated scour depth. • Localized removal and replacement of soft alluvial soils with adequately compacted structural fill. • Construction will be in creek. • Staged construction with Wall 15.
15	67½	11½[1]	Concrete Cantilever or MSE Wall	• Similar issues as presented above for Wall 14.
16	488	9	MSE	• Temporary cuts to install reinforcing may impact roadway, particularly at the east end of the wall. • Localized removal of soft soils and replacement with adequately compacted structural fill. • Relocate existing power poles. • Limited construction area. Likely require closure of one lane of traffic. Staged construction with Structure No. 2 and Walls 11 and 12.

Note:
[1] Heights extend to bottom of adjacent culvert foundations.

4.2.2 Standard Plan Reinforced Concrete Walls

Concrete cantilever retaining walls are readily installed without specialized equipment, are well-suited for fill applications, and are usually economical to construct up to heights of roughly 15 feet. The principal disadvantage of conventional concrete retaining walls is that a relatively large area must be available behind the wall for excavation of the temporary back-cut when the wall is used to support cut slopes. In addition, such walls are rigid and have relatively low tolerance for differential settlement.

WSDOT has developed Standard Plans (WSDOT, 2002) for concrete cantilever retaining walls. Wall Type 1 (level backslope) has been designed for the seismic conditions associated with western Washington and is suitable for Retaining Walls 11 to 15.

Section 15.5.1 of the WSDOT GDM states that these walls have been designed using Load Factor Design (LFD) per the AASHTO Standard Specifications for Highway Bridges. In our opinion, the Standard Plan Walls will exhibit adequate factors of safety with respect to global stability provided the walls are designed and constructed in accordance with the applicable WSDOT standards.

Retaining wall backfill materials should consist of "Common Borrow" as described in Section 9-03.14(3) of the WSDOT Standard Specifications. Common borrow will only be suitable for use as structural fill during dry weather. If wet weather construction is anticipated, the wall backfill material should consist of "Gravel Borrow" as described in Section 9-03.14(1) with the additional restriction that the fines content not exceed 5 percent. Placement and compaction of fill behind the walls should be in accordance with Section 2-09.3(1)E. Drainage behind the walls should be designed and constructed in accordance with WSDOT Standard Plan Sheet D-4.

Standard wall foundations should bear on dense native glacial soils, or densely compacted fill. New fill materials placed below wall foundation bearing levels should consist of "Class A Foundation Material" in accordance with WSDOT 9-03.17, as it is likely this fill will be placed in the presence of water from the existing creek. It may be necessary to place a separator geotextile fabric if the exposed subgrade soils are saturated, fine grained materials. The "Class A Foundation Material" should be compacted to at least 95 percent of the maximum dry density in accordance with WSDOT 2-03.3(14)C. Wall footing subgrades should be properly prepared, with the removal of all soft/loose or otherwise disturbed soil.

For Load and Resistance Factor Design (LRFD) design, Service, Strength and Extreme limit state bearing capacities for standard wall foundations are provided in Appendix D. Service limit state capacities are provided assuming approximately ½ inch and 1 inch of settlement will occur.

4.2.3 MSE Walls

4.2.3.1 General

Mechanically stabilized earth retaining walls are often a cost-effective method for support of fill embankments. MSE walls consist of alternating layers of backfill soil and reinforcing material with facing elements. Commonly used reinforcing materials include steel strips and various geosynthetic products such as geogrid and geotextile sheets. If geosynthetic products are selected, long term creep characteristics should be taken into consideration in product selection. The vertical spacing of the reinforcing elements is typically on the order of 1 to 3 feet, depending on the reinforcing material specified and other parameters. Pre-cast concrete members (panels or blocks) are widely used as facing elements. Design of an MSE wall system must be based on site-specific conditions and geotechnical parameters.

Principal advantages of MSE walls include relatively low unit cost and tolerance of relatively large differential settlements. In our opinion, an MSE wall system is suitable for Walls 8 and Walls 10 through 16.

Many MSE wall systems are available as proprietary wall systems. Ten proprietary MSE systems have been preapproved by WSDOT, as indicated in Appendix 15-D of the GDM. These wall systems are preapproved for heights up to 33 feet, and soil surcharge slopes above the wall, provided such slopes are 2H:1V or flatter.

4.2.3.2 General MSE Wall Design Parameters

We recommend the general design parameters summarized below in Table 5 for use in design of MSE walls. Design parameters specific to a particular wall location are presented in **Sections 4.2.3.3** through **4.2.3.7**. The values shown below assume the backfill soils and the retained soil are compacted to 95 percent of the maximum dry density in accordance with Method C, Section 2-03.3(14)C of the WSDOT Standard Specifications. Wall backfill within the reinforced zone should consist of "Gravel Borrow" as described in Section 9-03.14(1) of the WSDOT Standard Specifications.

MSE walls should also incorporate a drainage system to control water infiltration into the fill materials. The drainage system should be designed and constructed in accordance with the manufacturers' recommendations. As a minimum, we recommend the drainage system include an 18-inch thickness of "Gravel Backfill for Walls," WSDOT 9-03.12(2), placed behind the reinforced zone and in front of the existing or new fill embankment (retained soils). The drainage layer should extend to a 6 inch minimum diameter perforated drain pipe embedded in "Gravel Backfill for Drains," WSDOT 9-03.12 (4). The drain pipe should be graded to direct water into a storm drain system or other suitable outlet.

Table 5. Recommended Design Parameters for MSE Walls

Soil Properties	Backfill Soil		Retained Soil	Foundation Bearing Soil		
	Reinforced Zone[1]	Drainage Zone[2]	Existing or New Embankment Fill	Alluvium	Existing Fill	Glacial Till/ Advance Outwash
Unit Weight (pcf)	125	125	125	110	125	130
Friction Angle (deg)	38	36	36	34	36	40
Cohesion (psf)	0	0	0	0	0	0
Allowable Bearing Capacity (ksf)				2.5	2.5	6.0

Notes:
[1] Fill placed in the reinforced zone should consist of "Gravel Borrow" in accordance with WSDOT 9-03.14(1).
[2] Drainage materials placed between the reinforced zone and the retained soils should consist of "Gravel Backfill for Walls" in accordance with WSDOT 9-03.12(2).

Global static slope stability for MSE wall systems should have a factor of safety of 1.5 for walls supporting a structure and a factor of safety of 1.3 for walls not supporting another structure. Our slope stability analyses indicate adequate global stability for the proposed walls. Table 6 below summarizes the minimum factors of safety from our analyses for the static, seismic (pseudo static), and liquefied (static analysis with residual soil parameters) conditions. The global stability analyses are based on 2 feet of wall embedment below the ground surface. For walls where scour is possible (the head walls for Structure Nos. 2 and 3 and where culverts penetrate other wall locations), the planned embedment is 4 feet to account for 2 feet of potential scour. Illustrations showing critical failure surfaces and the slope stability analysis are presented in Appendix C.

Table 6. Factors of Safety for Global Stability

	Static	Seismic	Liquefaction
Wall 8	1.8	1.5	1.4
Wall 10	1.5	1.2	1.4
Wall 11	2.0	1.6	n/a
Wall 12	2.6	1.8	n/a
Wall 13	2.2	1.7	n/a
Wall 14	2.2	1.7	n/a
Wall 15	2.0	1.5	n/a
Wall 16	1.9	1.4	n/a

MSE walls should be designed with a factor of safety of 1.5 for sliding and pullout of reinforcing elements and 2 for overturning. If proprietary wall systems are used, the wall supplier is responsible for evaluating these items. However, we recommend that proprietary wall system designs be reviewed by a qualified geotechnical engineer, to verify that valid assumptions were made relative to material properties and other factors.

If the MSE wall will be subjected to the influence of surcharge loading (for example, traffic loading) within a horizontal distance equal to the height of the wall, the wall should be designed for the additional horizontal pressure using an appropriate design method. A common practice is to assume a surcharge loading equivalent to 2 feet of additional fill to simulate traffic loading; we consider this method appropriate for typical situations. If large surcharge loads such as from heavy trucks, cranes, or other construction equipment are anticipated in close proximity to the retaining wall, the wall should also be designed to accommodate the additional lateral pressures resulting from these concentrated loads.

4.2.3.3 Wall 8

Design Considerations. Wall 8 is located on the east side of SR 305 just south of Forest Rock Lane NE. The wall is approximately 1,150 feet long with a maximum height of about 16 feet. The location of Wall 8 is shown on the Site Plan, Figure 2B, and a plan and elevation view of the wall is presented on Figure 5. Figures 6 and 7 show our interpretation of the subsurface conditions based on current and historical explorations. The subsurface conditions, discussed previously in **Section 3.4.3**, generally consist of fill and alluvium overlying transitional bed deposits.

Due to the presence of relatively thick alluvial deposits which will experience consolidation, we recommend an MSE wall be constructed to support embankment fill for the widening of SR 305. Based on our analysis, we anticipate that settlement due to consolidation will be on the order of 3 to 6 inches. The thickness of the alluvium along the wall alignment is significantly less at the north end of the wall when compared to the south end of the wall. Therefore, differential settlement along the wall due to consolidation of the alluvium is likely. We anticipate that differential settlement along 25 feet of the wall could be as much as 2 inches. This is particularly likely in the middle portion of the wall where the thick alluvial deposits transition to a relatively thin layer. However, in our opinion, the estimated total and differential settlements are within the range that can be readily accommodated by MSE walls and two-stage construction (applying the facing elements after wall settlement is essentially complete) is not necessary.

Liquefiable soils are present within the alluvial deposits. We anticipate that settlement due to liquefaction will be on the order of 1 to 2 inches. The liquefaction-induced settlement will be in addition to the consolidation settlement described above.

We recommend a minimum embedment of 2 feet for Wall 8 in accordance with AASHTO and as specified in Section 15.4.2.5 of the GDM. Due to the presence of the existing creek near the proposed alignment of the wall, it is likely that soft wet soils will be present near foundation subgrade level. We recommend 2 feet of overexcavation below foundation subgrade where soft/wet alluvial soils are present. A geotextile separator fabric should be placed over the native soils and the structural fill should consist of "Class A Foundation Material." Scour was generally not considered in our analyses as we understand the creek will be relocated away from the face of the wall as part of this project. However, scour should be considered where culverts will penetrate the wall. Based on discussions with WSDOT, we understand scour up to 2 feet is assumed. Therefore we recommend 4 feet of embedment (2 feet below assumed scour depth) for the wall sections extending 10 feet from either side of the culvert penetration.

Construction Considerations. Temporary cuts to install the wall reinforcing may impact the existing roadway, and existing light poles will need to be relocated. We recommend that temporary cut slopes not be steeper than 1H:1V (horizontal to vertical). If seepage or sloughing is observed, it may be necessary to flatten the temporary slopes.

South Fork Dogfish Creek will need to be relocated away from the wall during construction. Because of the presence of the creek and the wetland near the north end of the wall, the contractor should expect soft and muddy conditions in the creek and wetland area and should select equipment accordingly. Overexcavation and replacement of soils along the wall alignment is expected. We recommend that a geotechnical engineer evaluate the foundation subgrade as it is prepared and prior to the placement of structural fill to evaluate the subgrade and make recommendations based on foundation conditions present.

The wall construction will need to accommodate existing culverts that are present along the alignment of the proposed wall. The existing culverts along the wall consist of pipes ranging in size from 18 to 36 inches in diameter as well as two 103-inch by 71-inch corrugated steel pipe arches.

4.2.3.4 Wall 10

Design Considerations. Wall 10 is located on the east side of SR 305 just north of Forest Rock Lane NE. The wall is approximately 1,345 feet long with a maximum height of 13 feet. The location of Wall 10 is shown on the Site Plan, Figure 2C, and a plan and elevation view of the wall is presented on Figure 8. Figure 9 show our interpretation of the subsurface conditions based on current and historical explorations. The subsurface conditions, discussed previously in **Section 3.4.4**, generally consist of fill and alluvium overlying transitional bed deposits.

Due to the presence of alluvial deposits which vary in thickness along the wall alignment and are expected to experience consolidation, we recommend an MSE wall be constructed to support embankment fill for the widening of SR 305. Based on our analysis, we anticipate that settlement due to consolidation will be on the order of 2 to 4 inches. The thickness of the alluvium along the wall ranges from a few feet at the south end to about 10 feet near the middle of the wall. The depth of the alluvium at the north end of the wall is such that it will likely be removed in order to get to the foundation elevation. Therefore, differential settlement along the wall due to consolidation of the alluvium is likely. We anticipate that differential settlement along 25 feet of the wall could be as much as 2 inches. This is particularly likely where the thick alluvial deposits transitions to a relatively thin layer at the south end of the wall or where the wall is founded on transitional bed deposits at the north end. In our opinion, the estimated total and differential settlements are within the range that can be readily accommodated by MSE walls and two-stage construction is not necessary.

Liquefiable soils are present within the alluvial deposits. We anticipate that settlement due to liquefaction will be on the order of 1 to 2 inches. The liquefaction-induced settlement will be in addition to the consolidation settlement described above.

We recommend a minimum embedment of 2 feet for Wall 10 in accordance with AASHTO and as specified in Section 15.4.2.5 of the GDM. Due to the presence of the existing creek near the proposed alignment of the wall, it is likely that soft wet soils will be present near foundation subgrade level. We recommend 2 feet of overexcavation below foundation subgrade where soft/wet alluvial soils are present. A geotextile separator fabric should be placed over the native soils and the structural fill should consist of "Class A Foundation Material." Scour was generally not considered in our analyses as we understand the creek will be relocated away from the face of the wall as part of this project. However, scour should be considered where culverts will penetrate the wall. Based on discussions with WSDOT, we understand scour up to 2 feet is assumed. Therefore we recommend 4 feet of embedment (2 feet below assumed scour depth) for the wall sections extending 10 feet from either side of the culvert penetration.

Construction Considerations. Temporary cuts to install the wall reinforcing may impact the existing roadway. We recommend that temporary cut slopes not be steeper than 1H:1V (horizontal to vertical). If seepage or sloughing is observed, it may be necessary to flatten the temporary slopes.

South Fork Dogfish Creek will need to be relocated away from the wall during construction. Because of the presence of the creek, the contractor should expect soft and muddy conditions in the creek area and should select equipment accordingly. Overexcavation and replacement of soils along the wall alignment is expected. We recommend that a geotechnical engineer evaluate the foundation subgrade as it is prepared and prior to the placement of crushed rock to evaluate the subgrade and make recommendations based on foundation conditions present.

The wall construction will need to accommodate an existing 18-inch diameter pipe located near the north end of the wall.

4.2.3.5 Wall 13

Design Considerations. Wall 13 is located at the northeast quadrant of the intersection of NE Lincoln Road and 8th Avenue NE. The wall is approximately 71 feet long with a maximum height of about 6 feet. The location of Wall 13 is shown on the Site Plan, Figure 2A, and a plan and elevation view of the wall is presented on Figure 13. Figure 14 show our interpretation of the subsurface conditions based on current and historical explorations. The subsurface conditions, discussed previously in **Section 3.4.6**, generally consist of fill overlying advance outwash. The wall will be the headwall for an 8-ft diameter CMP culvert.

We recommend a minimum embedment of 2 feet for Wall 13 in accordance with AASHTO and as specified in Section 15.4.2.5 of the GDM. Due to the presence of the creek near the proposed alignment of the wall, it is likely that soft wet soils will be present near foundation subgrade level. We recommend that saturated soft soils and existing fill be removed and replaced with "Class A Foundation Material." The wall should be founded at least 2 feet below the elevation of the anticipated scour depth, which we understand to be 2 feet based on discussions with WSDOT.

Construction Considerations. Temporary cuts to install the wall reinforcing may impact the existing roadways. We recommend that temporary cut slopes not be steeper than 1H:1V (horizontal to vertical). If seepage or sloughing is observed, it may be necessary to flatten the temporary slopes.

Construction of the wall may occur concurrently with the installation of the 8-foot CMP culvert which will need to be incorporated into the wall design. In addition, if NE Lincoln Road is to remain open during construction, the wall and culvert improvements will need to be coordinated with construction of Wall 5.

4.2.3.6 Walls 11, 12, 14, 15

Walls 11, 12, 14, and 15 are grouped together because they will each function as culvert headwalls, have similar geometry and adjacent site features, and are anticipated to have similar subsurface conditions.

Design Considerations. Walls 11 and 12 are located on the north and south sides of Bond Road (SR 307), respectively, just east of the intersection with NE Bernt Road. The walls are 78 and 93 feet long, with maximum exposed heights of 12½ and 15 feet, respectively. The locations of Walls 11 and 12 are shown on the Site Plan, Figure 2E, and a plan and elevation view of the walls is presented on Figure 10. Figures 11 and 12 show our interpretation of the subsurface conditions based on current and historical explorations. The subsurface conditions, discussed previously in **Section 3.4.5**, generally consist of fill overlying glacial till and advance outwash. The walls will be the headwalls for 3-sided Structure No. 2.

Walls 14 and 15 are located on the north and south sides of Bond Road (SR 307), respectively just east of the intersection with 1st Avenue NE. The walls are approximately 46 and 67 feet long, with maximum exposed heights of 10 and 11½ feet, respectively. The locations of Walls 14 and 15 are shown on the Site Plan, Figure 2D, and a plan and elevation view of the walls is presented on Figure 15. Figures 16 and 17 show our interpretation of the subsurface conditions based on current and historical explorations. The subsurface conditions, discussed previously in **Section 3.4.7**, generally consist of fill overlying advance outwash and transitional deposits. The walls will be the headwalls for 3-sided Structure No. 3.

We recommend that the minimum embedment for the wall be in accordance with AASHTO, as specified in Section 15.4.2.5 of the GDM. We understand that it is typical WSDOT practice to extend the base of wall to the foundation elevation of the culvert structures. We recommend that as a minimum, the wall be founded at least 2 feet below the anticipated scour depth. We understand that WSDOT anticipates a scour depth of about 2 feet. Therefore, culverts and walls should be founded approximately 4 feet below existing grade. The additional depth for embedment and scour make prefabricated modular block walls, such as gabions, less desirable because of the total height of the walls.

Construction Considerations. The creek may need to be diverted or contained in pipes during construction and the contractor should expect localized soft and muddy conditions. Due to the presence of the creek near the proposed walls and possibility of deterioration of the foundation subgrade soils in the presence of the water, we recommend that structural fill used to replace loose wet soils consist of "Class A Foundation Material."

For each of the four walls, construction will likely occur at the same time as the construction of the culvert structures that cross below Bond Road. A partial road closure and staged construction will likely be required in order to construct the walls and culverts without closing the road completely. Temporary shoring will be necessary to perform the staged construction and make the necessary excavations to install the culverts and retaining walls within the roadway.

The existing rockery located along the south side of Bond Road should be removed as part of the construction of Wall 14. The construction of Wall 14 should accommodate existing orphan culverts that are present in the vicinity of this wall.

Wall 15 will need to be designed to accommodate the 44.4-inch diameter concrete pipe that extends from the storm system below Bond Road to the location of Wall 15 as well as an existing storm drain line located on the west side of the proposed wall. Temporary shoring used to construct the walls and/or culverts will need to be designed to accommodate existing storm drain utilities in the vicinity of Walls 14 and 15.

4.2.3.7 Wall 16

Design Considerations. Wall 16 is located on the south side of Bond Road just east of NE Bernt Road. The wall is approximately 387 feet long with a maximum exposed height of about 9 feet. The location of Wall 16 is shown on the Site Plan, Figure 2E, and a plan and elevation view of the wall is presented on Figure 18. Figure 19 show our interpretation of the subsurface conditions based on current and historical explorations. The subsurface conditions, discussed previously in **Section 3.4.8**, generally consist of fill overlying advance outwash. We understand that an MSE wall is anticipated for Wall 16, which will support an embankment fill for the widening of Bond Road.

We recommend a minimum embedment of 2 feet for Wall 16 in accordance with AASHTO and as specified in Section 15.4.2.5 of the GDM. The wall should be founded on dense undisturbed advance outwash soils. If localized areas of loose or wet soils are observed at the foundation subgrade. We recommend these areas be recompacted if possible or replaced with "Class A Foundation Material."

The existing fill is thickest at the east end of the wall. If excavations to remove the fill and reach advance outwash soils become excessive, the wall should be founded on at least 2 feet of new structural fill. We recommend that where the wall is to be founded over existing fill, 2 feet of the existing fill be removed and replaced with adequately compacted structural fill. If the remove and replace option is selected, we recommend an allowable bearing pressure of 2.5 ksf. Assuming that the wall foundation subgrade is adequately prepared, we anticipate that settlement will be on the order of 1 to 2 inches. Differential settlement along 25 feet of wall will be on the order of 1 inch.

Construction Considerations. Temporary cuts to install the wall reinforcing may impact the existing roadway, particularly at the east end of the wall. There is limited construction area and one lane of traffic will likely be closed. Construction of the wall will need to be coordinated with construction of Walls 11 and 12 and Structure No. 2 that will cross below Bond Road. In addition, the existing power poles along the proposed wall alignment will need to be relocated.

4.2.4 Cantilever Soldier Pile Wall

4.2.4.1 General

Wall 5 is located within the southwest quadrant of the intersection of NE Lincoln Road with SR 305; the wall is approximately 165 feet long with a maximum exposed height of 16 feet. The location of Wall 5 is shown on the Site Plan, Figure 2A, and a plan and elevation view of the wall is presented on Figure 3. Figure 4 show our interpretation of the subsurface conditions based on current and historical explorations. The wall will be constructed along the existing embankment fill and will be used to retain additional fill for the widening of SR-305. The subsurface conditions, discussed previously in **Section 3.4.2**, generally consist of fill and alluvium overlying advance outwash.

We understand there are several underground utilities along NE Lincoln Road and SR 305 near the planned location of Wall 5. It is the desire of WSDOT not to relocate these utilities, therefore a wall alternative that will not require the relocation of the utilities is planned. Temporary cut slopes to install an MSE wall would likely require the relocation of the underground utilities. Therefore, WSDOT has decided to construct a cantilever soldier pile wall to retain the proposed fill.

Cantilever soldier pile walls typically consist of steel beams that are concreted into drilled vertical holes located along the wall alignment. Soldier piles are typically spaced 6 to 8 feet apart. Once the vertical beams have been installed, chemically treated timber or shotcrete lagging is typically placed behind the flanges of the steel beams to retain the soil placed behind the soldier piles. A concrete fascia is often placed in front of the lagging for permanent walls. The existing slope, creek and the presence of overhead utilities will need to be considered during construction of the soldier pile wall.

We understand that the new 3-sided open bottom culvert, which will be installed to replace the existing culvert below SR 305, will daylight through the proposed soldier pile wall. In addition, we understand that an 8-foot corrugated metal pipe (CMP) is planned for the creek crossing below NE Lincoln Road. The soldier piles on either side of the culvert and CMP will have to be spaced and designed accordingly to accommodate these structures.

4.2.4.2 Design Considerations

An earth pressure diagram for the proposed soldier pile wall is presented in Appendix E. The active lateral earth pressures above the bottom of the wall should be assumed to act across the soldier pile spacing as it is composed of the load transferred from the lagging to the piles. The active earth pressure below the bottom of the excavation for the static condition is assumed to act over the soldier pile diameter. The passive earth pressures below the bottom of the wall are assumed to act over 3 soldier pile diameters or the soldier pile spacing, whichever is less. As shown on the lateral earth pressure diagram, we recommend disregarding passive resistance contribution from soils less than 2 feet below the bottom of the wall.

Wall drainage should be provided in accordance with Section 15.4.2.12 of the GDM. If a permanent concrete facing is cast against the lagging, a composite drainage material must be attached to the lagging prior to the casting of the permanent facing. If precast concrete panels are used as facing for the wall, it will not be necessary to install drainage material provided water can readily pass through the lagging.

As discussed above, the soldier pile wall will need to be designed to accommodate the open bottom culvert and the CMP that will both daylight through the wall. The proposed open bottom culvert is anticipated to have a span width of 10 feet. This width exceeds the typical 6- to 8-foot spacing of soldier piles. It will be necessary to design the soldier piles on either side of the culvert and CMP to accommodate the increased spacing and associated lateral earth pressures.

4.2.4.3 Construction Considerations

Access to install the soldier piles is anticipated to be difficult. The wall alignment is situated near the base of a moderately steep slope. The existing creek runs along the base of the slope and through existing culverts that will be replaced during this project. Existing underground utilities are located along the south side of NE Lincoln Road and along the west side of SR 305. In addition, several overhead utility lines are present along the south side of NE Lincoln Road.

Temporary casing or drilling fluid may be required to install the soldier piles because they will extend below the groundwater level. Cobbles and boulders are frequently encountered in glacial soils. Although not observed in our explorations, the surficial fill may also contain debris that could affect installation of the soldier piles. The contractor should be prepared to address the presence of fill, debris, boulders, and groundwater during construction.

4.3 CULVERTS

4.3.1 General

A 3-sided open bottom box culvert is planned for the South Fork Dogfish Creek crossing below SR 305 just south of the intersection with NE Lincoln Road. This culvert is planned to be about 107 feet in length with a 10 foot span and a height of about 12½ feet, which accounts for up to 2 feet of scour. Two additional culverts are planned along Bond Road. One of the culverts is just east of the intersection with 1st Avenue NE. This culvert is approximately 75 feet in length with a span width of 12 feet and a height of 13 feet, which accounts for up to 2 feet of scour. The other culvert will be located just east of Bernt Road NE. This culvert is about 80 feet in length with a span width of 16 feet and a height of 11 feet, which accounts for up to 2 feet of scour.

4.3.2 Bearing Capacity

We understand that the open bottom box culverts are being designed with the assumption that 2 feet of scour is likely to occur. We therefore recommend that the culvert footings be placed at least 2 feet below the elevation of the anticipated scour depth. Based on the explorations and available subsurface soil information, the three structures will be supported on dense advance outwash. We recommend that the geotechnical engineer evaluate the footing subgrade during construction to confirm that the subgrade soils are as anticipated.

We understand that the 3-sided culverts may be designed using either the LFD or LRFD approach. If the LFD approach is used, we recommend an allowable bearing capacity of 6 ksf for foundations bearing on undisturbed glacially consolidated soils. For LRFD design, Service, Strength and Extreme limit state bearing capacities for foundations bearing on undisturbed glacially consolidated soils are provided in Appendix D. Service limit state capacities, as a function of settlement and foundation width, are provided for ½ inch and 1 inch of settlement.

The creek may need to be diverted or contained in pipes during construction and the contractor should expect localized soft and muddy conditions. Due to the presence of the creek there exists the possibility of deterioration of the foundation subgrade soils in the presence of the water, we recommend that structural fill used to replace loose wet soils consist of "Class A Foundation Material."

We recommend that the resistance factors listed in Table 7 be used when evaluating the different limit states for the culvert foundations.

Table 7. Recommended Resistance Factors for Culvert Foundations

Limit State	Bearing	Shear Resistance to Sliding	Passive Pressure Resistance to Sliding
Strength	0.45	0.8	0.5
Service	1	1	1
Extreme	.9	.9	.9

4.3.3 Construction Considerations

Construction of the culverts will likely occur at the same time as the construction of the adjacent retaining walls. Partial road closures and staged construction will likely be required in order to construct the walls and culverts without closing the road completely. Temporary shoring along the roadway will be necessary to perform the staged construction while keeping at least one lane of traffic open.

Caving soils should be anticipated within the saturated advance outwash soils. The contractor should be prepared to deal with groundwater issues and caving soil conditions.

Although not observed in our explorations, cobbles and boulders are frequently encountered in glacially derived soils. The contractor should be prepared to deal with cobbles and boulders that may present themselves during the excavation process or during the installation of temporary shoring.

The creek may need to be diverted or contained in pipes during construction of the new culverts. Access to the base of the existing creek may be difficult because of the relatively steep embankment slopes. In addition, overhead utilities are present along the south side of Bond Road (SR-307).

4.4 RECOMMENDED ADDITIONAL SERVICES

Because the future performance and integrity of the structural and geotechnical elements of this project will depend largely on proper Plans, Specifications, and Estimate (PS&E) preparation and diligent construction procedures, we recommend that the Geotechnical Engineer provide the following post-report services:

- The Geotechnical Engineer should prepare the Summary of Geotechnical Conditions to be included in the PS&E as an appendix. The summary should be prepared as part of the PS&E review process.
- The Geotechnical Engineer should review all construction plans and specifications to verify that the design criteria presented in this report have been interpreted correctly and properly integrated into the design.
- The Geotechnical Engineer should attend pre-construction conferences with the Construction Project Engineer and Contractor to discuss important geotechnically related construction issues.
- The Geotechnical Engineer should review Contractor submittals for all temporary and permanent shoring walls, retaining walls, and other geotechnically challenging elements of the project.
- The Geotechnical Engineer should observe exposed subgrades for culvert foundations and retaining walls after completion of stripping and excavation to contract elevations to confirm that suitable soil conditions have been reached and to determine appropriate subgrade preparation methods.

5.0 LIMITATIONS

We have prepared this report for the exclusive use by WSDOT for the SR 305, OL-3420 Poulsbo SCL to Bond Road project. The data and report should be provided to prospective contractors for their bidding or estimating purposes, but our report, conclusions and interpretations should not be construed as a warranty of the subsurface conditions.

Within the limitations of scope, schedule and budget, our services have been executed in accordance with generally accepted practices in the fields of geotechnical engineering in this area at the time this report was prepared. No warranty or other conditions, express or implied, should be understood.

Any electronic form, facsimile or hard copy of the original document (email, text, table, and/or figure), if provided, and any attachments are only a copy of the original document. The original document is stored by GeoEngineers, Inc. and will serve as the official document of record.

Please refer to Appendix F titled "Report Limitations and Guidelines for Use" for additional information pertaining to use of this report.

6.0 REFERENCES

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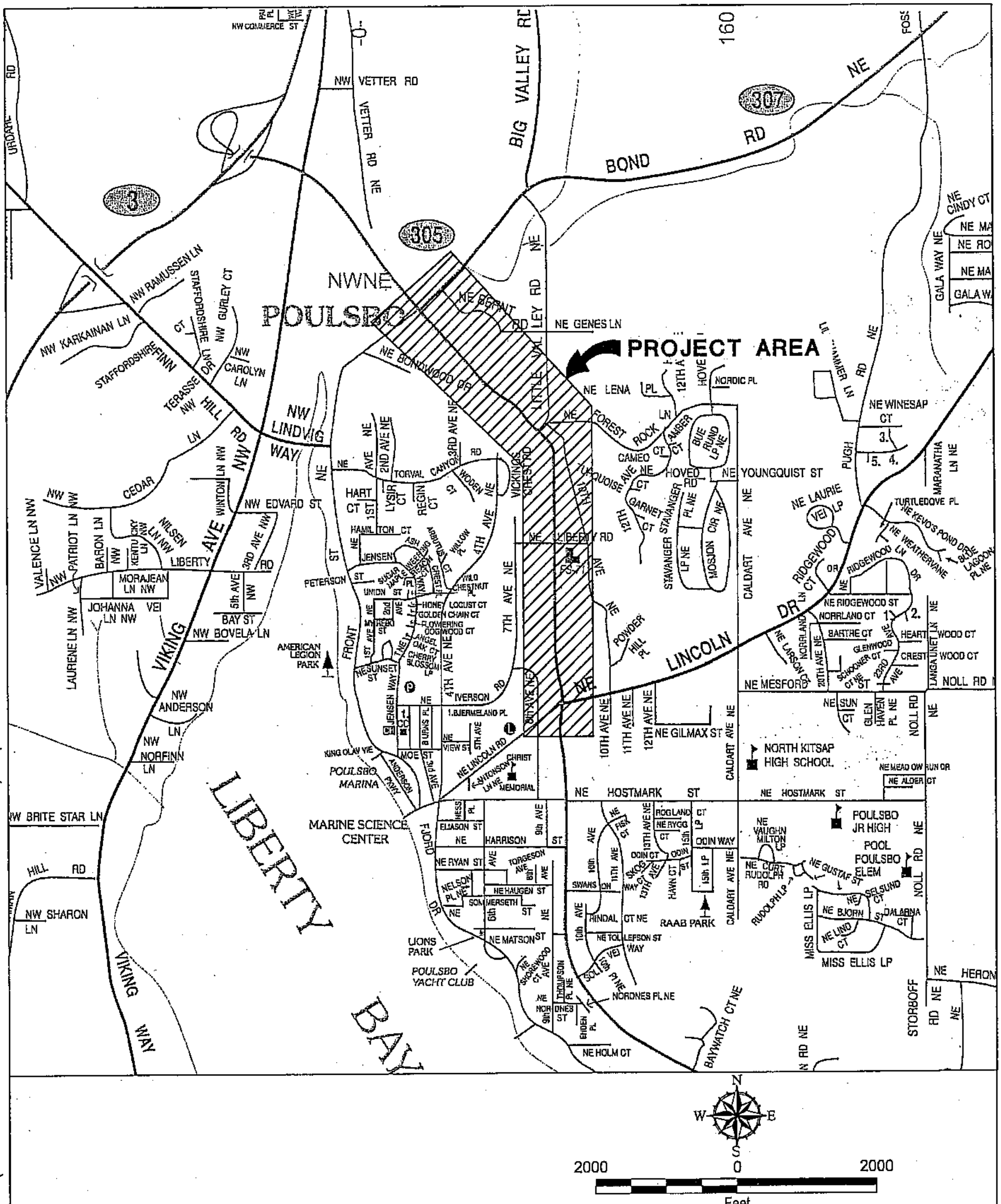

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Vicinity Map

SR 305, OL-3420

Poulsbo SCL to Bond Road Improvements

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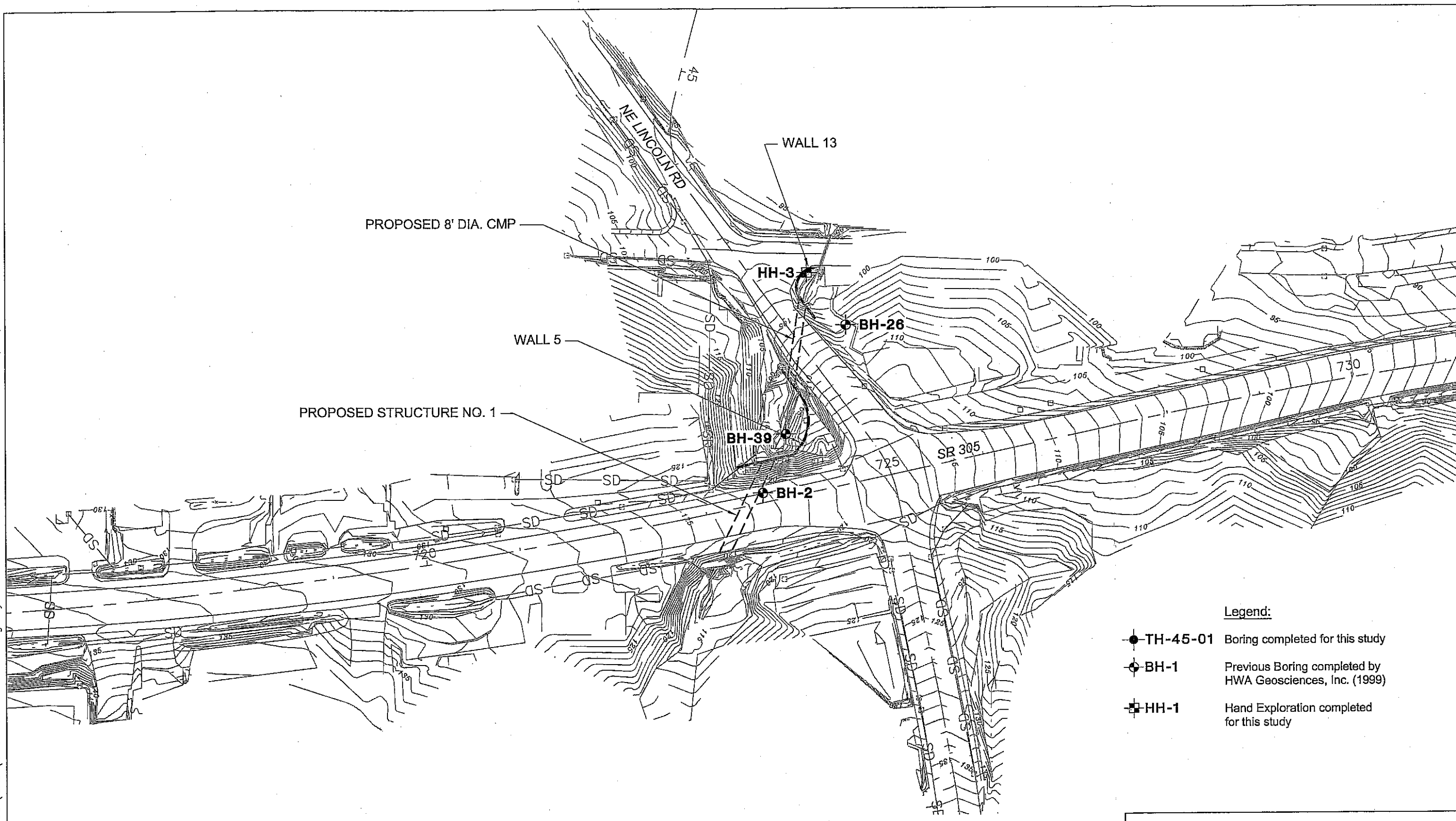
Figure 1



Notes:

1. The subsurface conditions shown are based on interpolation between widely spaced explorations and should be considered approximate; actual subsurface conditions may vary from those shown.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record.

Reference: Drawing entitled "SR 305, Lincoln Rd. to Bond Rd. - HOV Lanes, Retaining Wall Details" by Washington State Department of Transportation received 06/21/05.

Site Plan

SR 305, OL-3420

Poulsbo SCL to Bond Road Improvements

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Figure 2A

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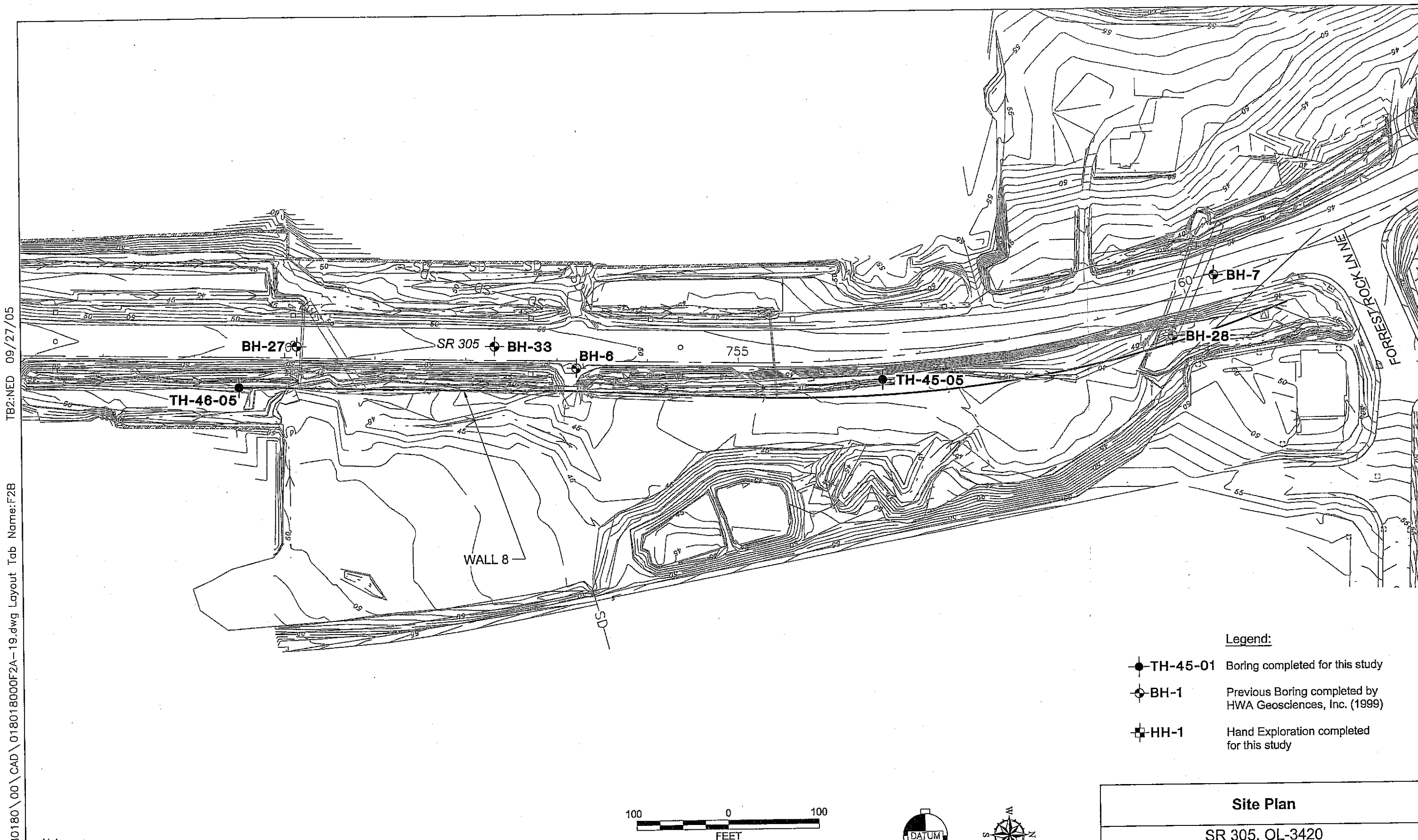

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Notes:
1. The subsurface conditions shown are based on interpolation between widely spaced explorations and should be considered approximate; actual subsurface conditions may vary from those shown.
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Site Plan

SR 305, OL-3420
Poulsbo SCL to Bond Road Improvements

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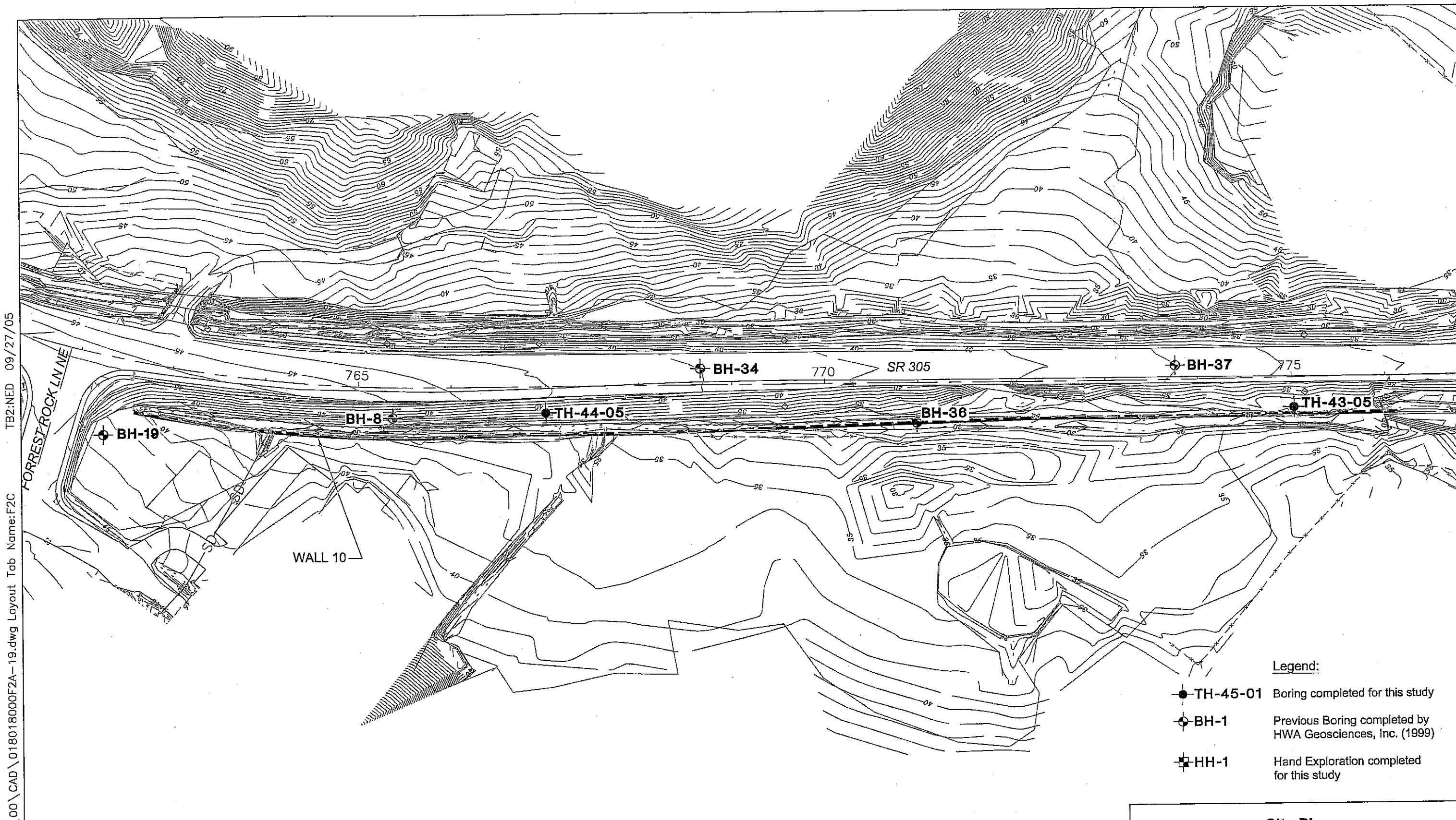
Figure 2B



Legend:

- TH-45-01 Boring completed for this study
- BH-1 Previous Boring completed by HWA Geosciences, Inc. (1999)
- HH-1 Hand Exploration completed for this study

Notes:
1. The subsurface conditions shown are based on interpolation between widely spaced explorations and should be considered approximate; actual subsurface conditions may vary from those shown.
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Site Plan

SR 305, OL-3420
Poulsbo SCL to Bond Road Improvements

GEOENGINEERS

Figure 2C

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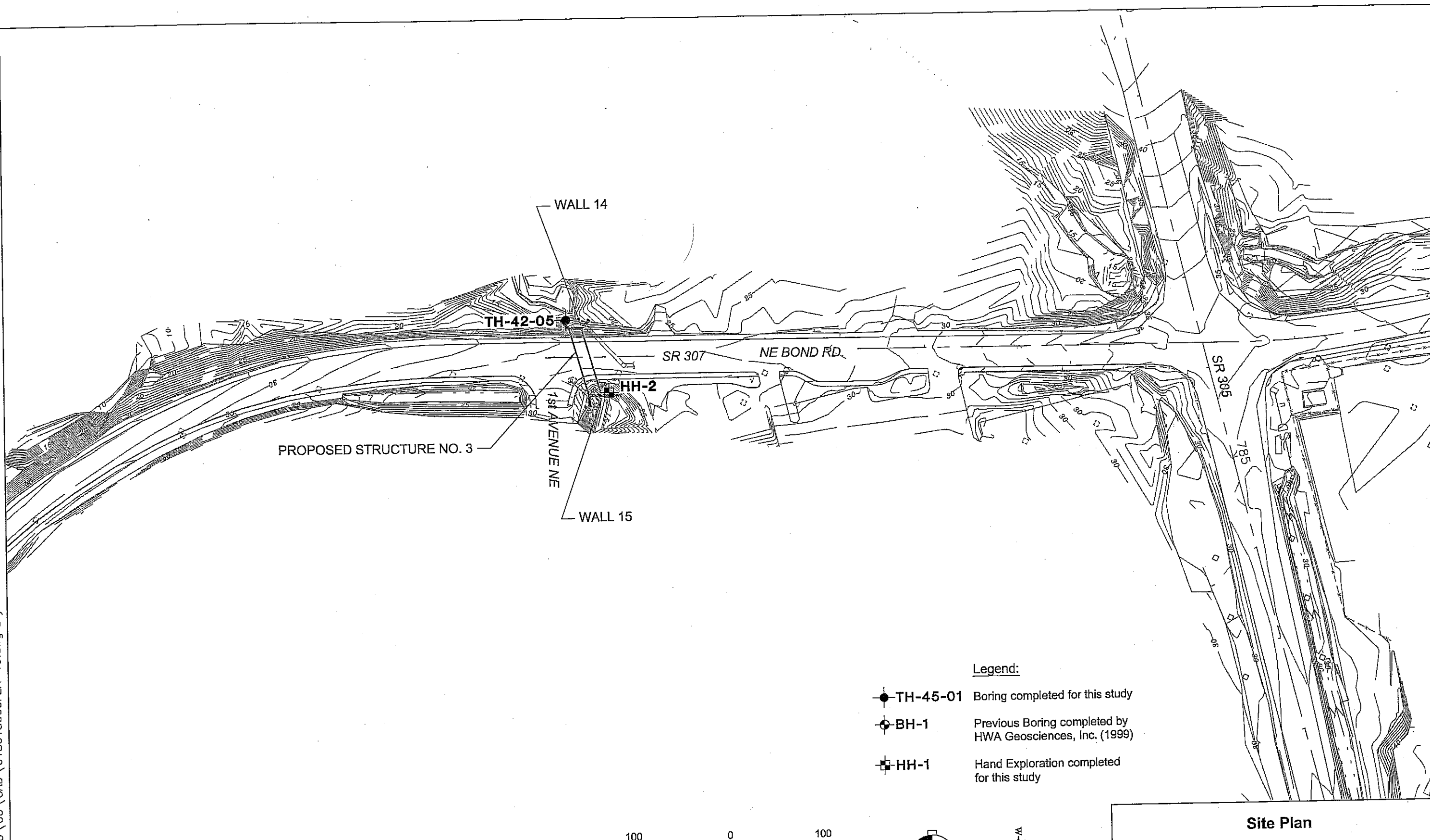

Notes:
1. The subsurface conditions shown are based on interpolation between widely spaced explorations and should be considered approximate; actual subsurface conditions may vary from those shown.
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Site Plan

SR 305, OL-3420
Poulsbo SCL to Bond Road Improvements

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Figure 2D

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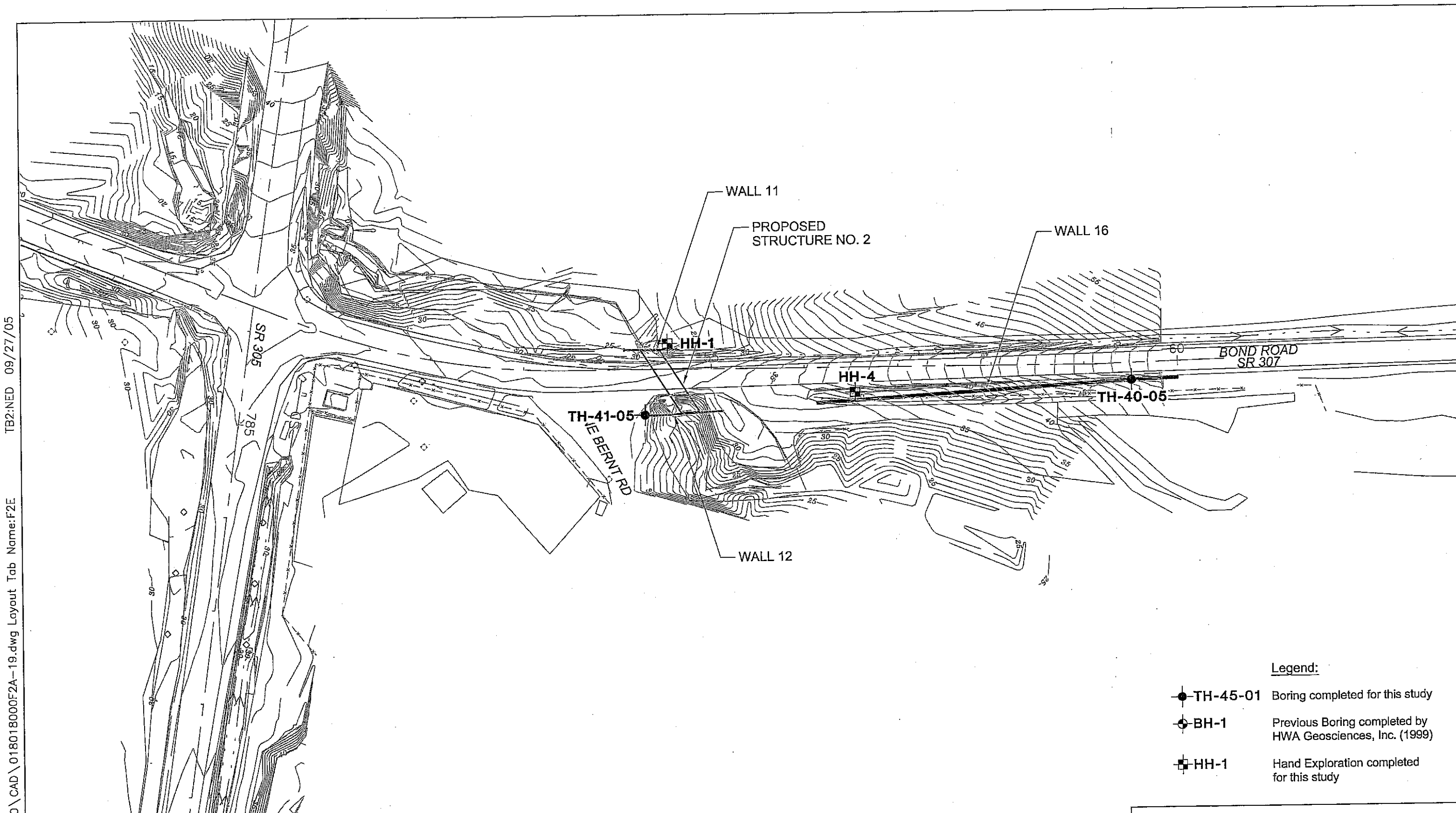

Legend:

- TH-45-01 Boring completed for this study
- BH-1 Previous Boring completed by HWA Geosciences, Inc. (1999)
- HH-1 Hand Exploration completed for this study

100 0 100
FEET

DATUM
(NAVD) 88

Notes:
1. The subsurface conditions shown are based on interpolation between widely spaced explorations and should be considered approximate; actual subsurface conditions may vary from those shown.
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Site Plan

SR 305, OL-3420
Poulsbo SCL to Bond Road Improvements

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Figure 2E

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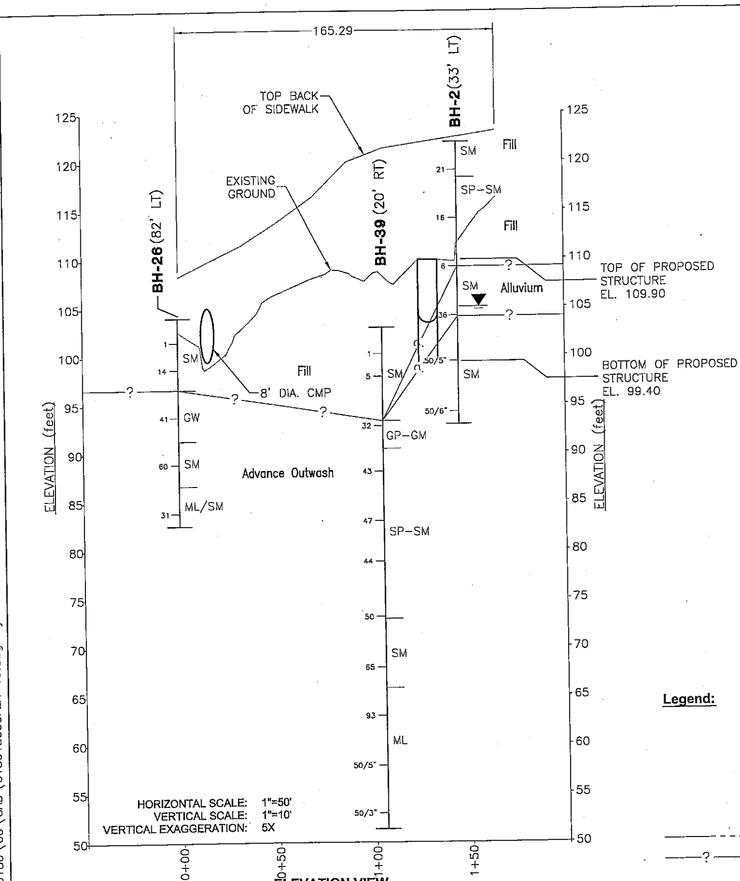

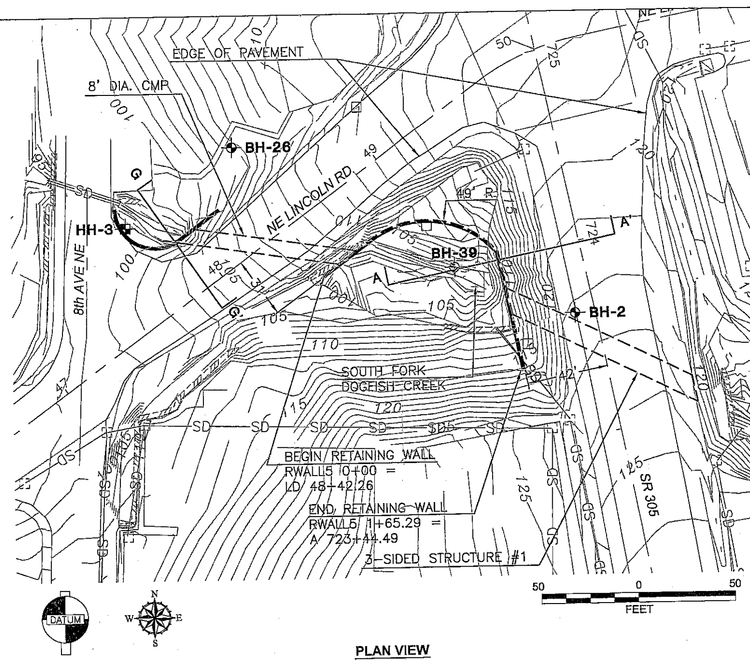

Legend:

B-1 (10ft LT)

- Offset from Section/Elevation View (Distance, Direction)
- Boring Number
- SM Soil Classification
- Soil Contact
- Groundwater/Seep Level
- Inferred Soil Contact
- 47 Blow Count
- SH Shelby Tube Sample

Legend:

- BH-1 Previous Boring completed by HWA Geosciences, Inc. (1999)
- HH-1 Hand Exploration completed for this study

Retaining Wall 5
Plan & Elevation

SR 305, OL-3420
Poulsbo SCL to Bond Road Improvements

GEOENGINEERS

Figure 3

Notes:
1. The subsurface conditions shown are based on interpolation between widely spaced explorations and should be considered approximate; actual subsurface conditions may vary from those shown.
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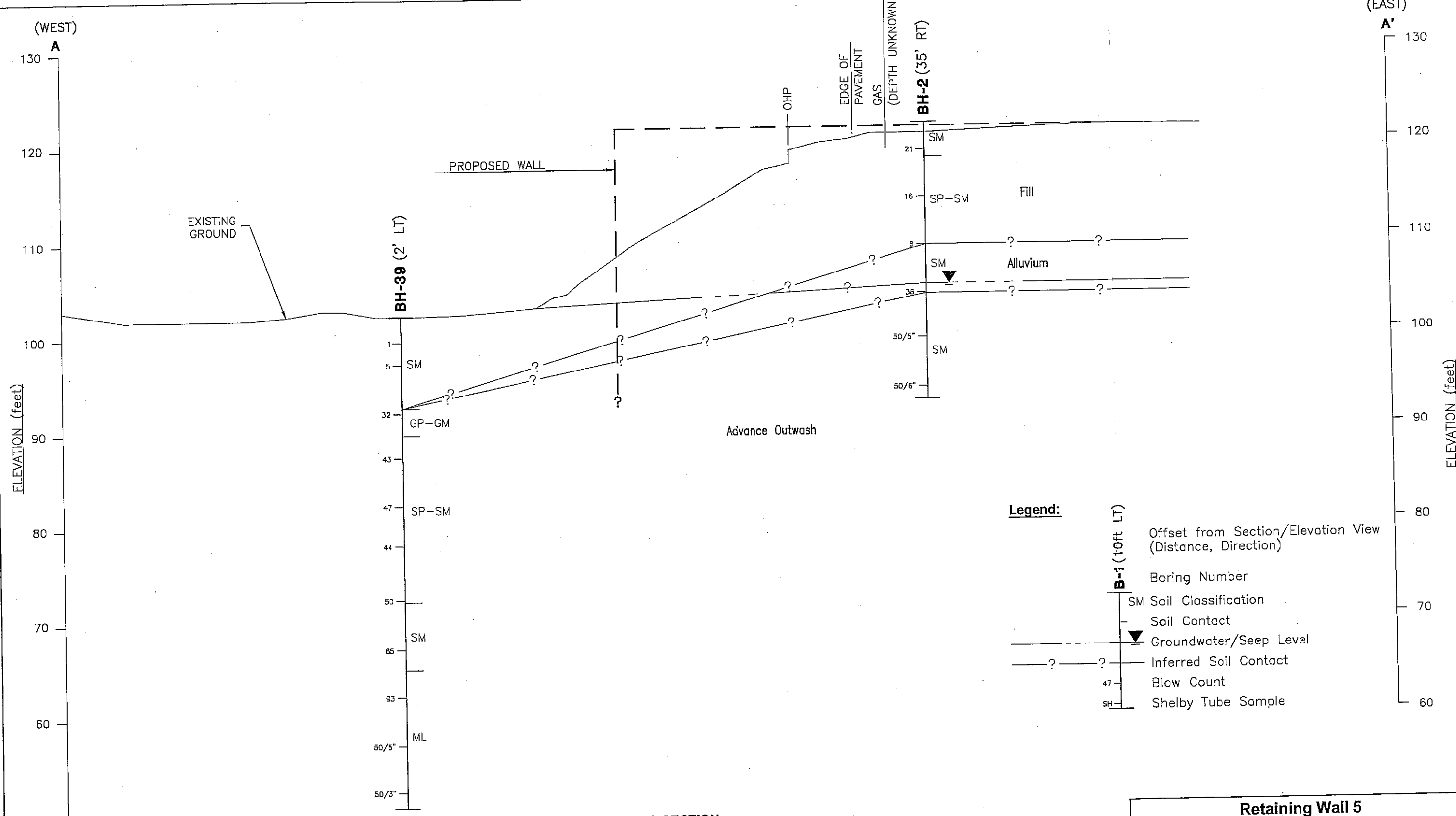

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CROSS SECTION

HORIZONTAL SCALE: 1"=10'
VERTICAL SCALE: 1"=10'

Notes:
1. The subsurface conditions shown are based on interpolation between widely spaced explorations and should be considered approximate; actual subsurface conditions may vary from those shown.
2. Refer to Figure 3 for location of Section A-A'.
3. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record.

Reference: Cross Section developed by GeoEngineers, September 2005.

Retaining Wall 5
Cross Section A-A'

SR 305, OL-3420
Poulsbo SCL to Bond Road Improvements

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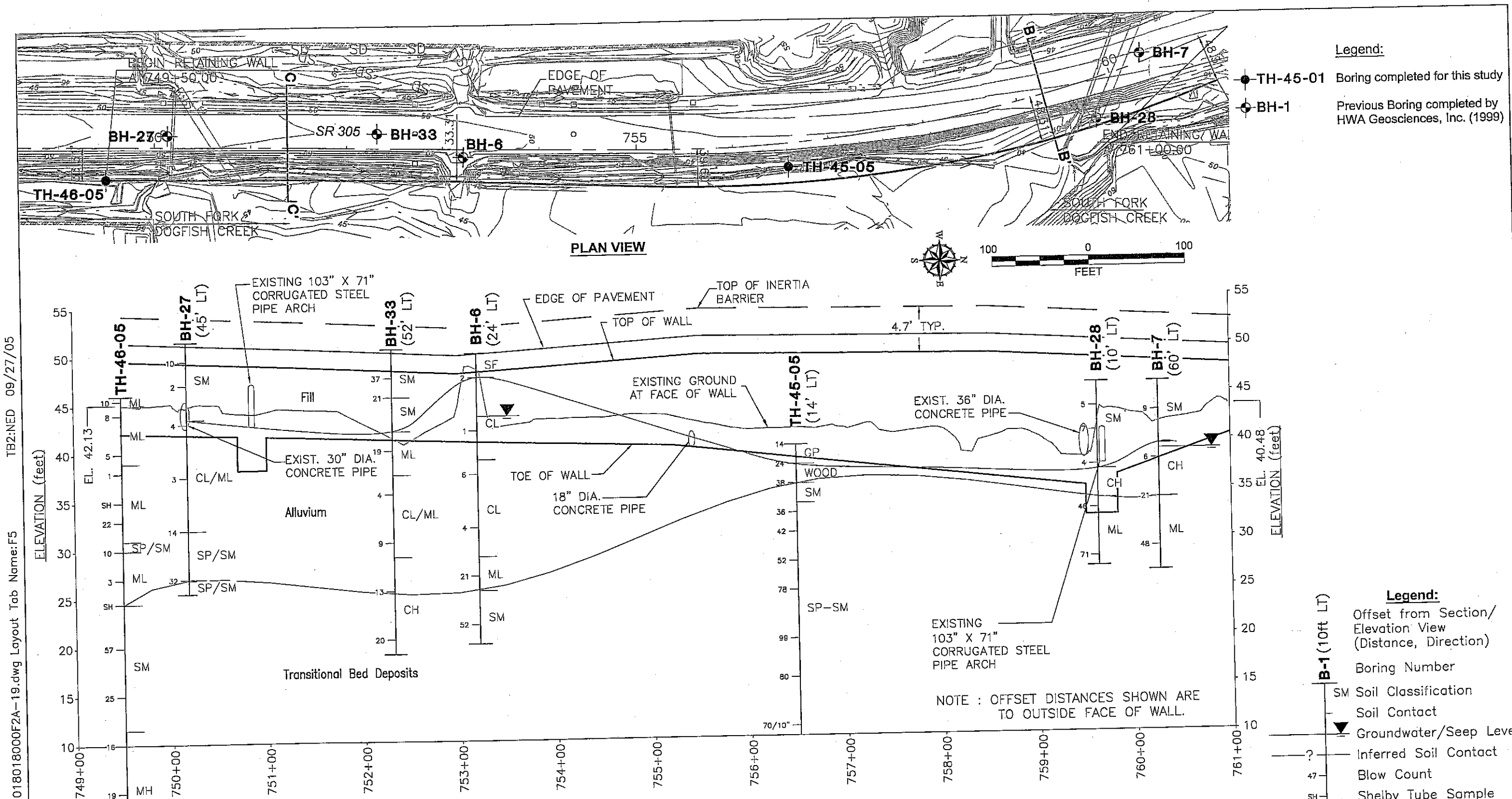
Figure 4



ELEVATION VIEW

HORIZONTAL SCALE: 1"=100'
VERTICAL SCALE: 1"=10'
VERTICAL EXAGGERATION: 10X

DATUM (NAVD) 88

Notes:
1. The subsurface conditions shown are based on interpolation between widely spaced explorations and should be considered approximate; actual subsurface conditions may vary from those shown.
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Retaining Wall 8
Plan & Elevation

SR 305, OL-3420
Poulsbo SCL to Bond Road Improvements

GEOENGINEERS | **Figure 5**

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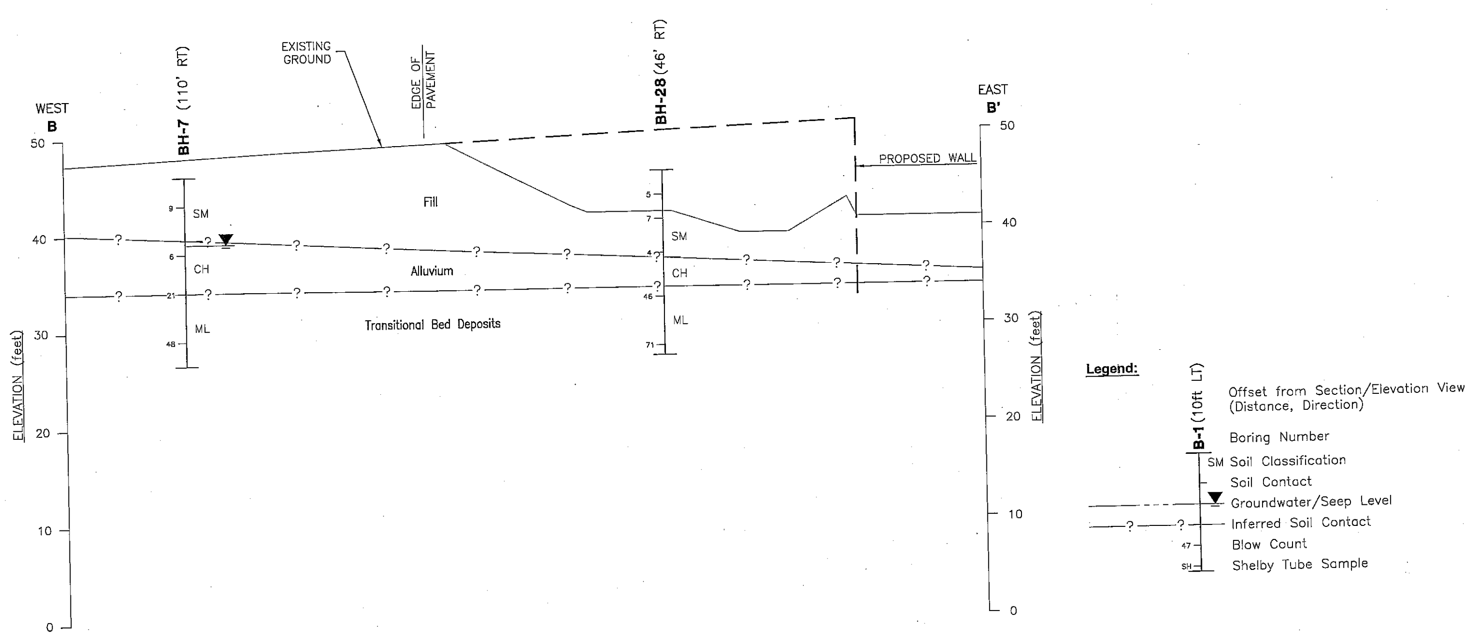

CROSS SECTION
HORIZONTAL SCALE: 1"=10'
VERTICAL SCALE: 1"=10'

Notes:
1. The subsurface conditions shown are based on interpolation between widely spaced explorations and should be considered approximate; actual subsurface conditions may vary from those shown.
2. Refer to Figure 5 for location of Section B-B'.
3. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record.

Reference: Cross Section developed by GeoEngineers, September 2005.

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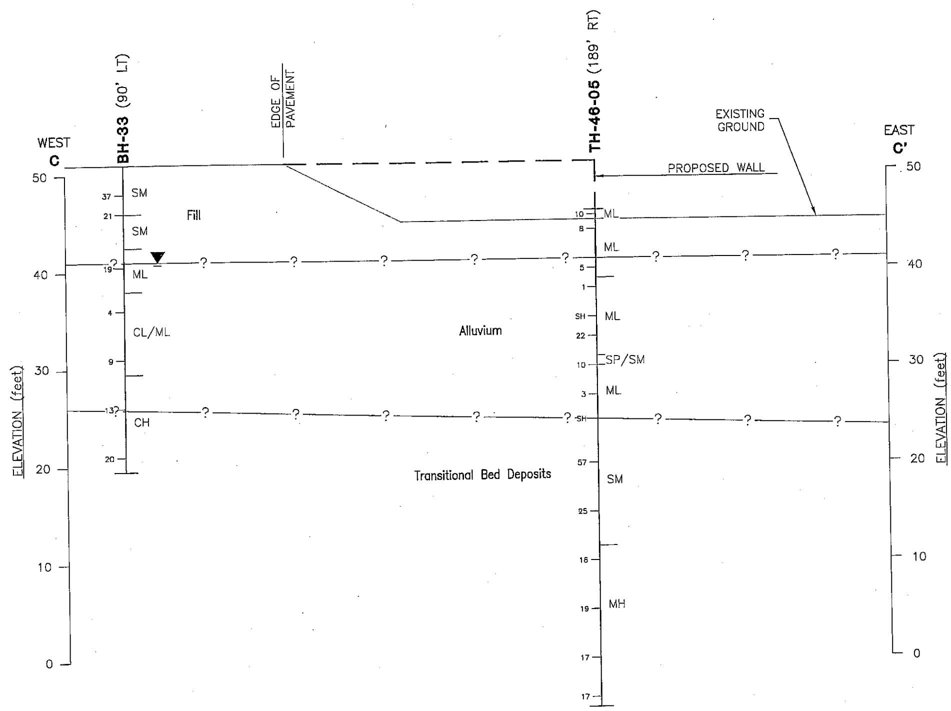
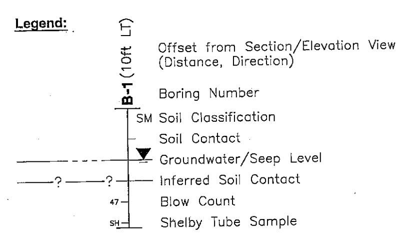
Retaining Wall 8
Cross Section B-B'
SR 305, OL-3420
Poulsbo SCL to Bond Road Improvements
GEOENGINEERS
Figure 6



CROSS SECTION
HORIZONTAL SCALE: 1"=10'
VERTICAL SCALE: 1"=10'

Legend:



Notes:
1. The subsurface conditions shown are based on interpolation between widely spaced explorations and should be considered approximate; actual subsurface conditions may vary from those shown.
2. Refer to Figure 5 for location of Section C-C'.
3. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record.
Reference: Cross Section developed by GeoEngineers, September 2005.

Retaining Wall 8
Cross Section C-C'
SR 305, OL-3420
Poulsbo SCL to Bond Road Improvements
GEOENGINEERS
Figure 7

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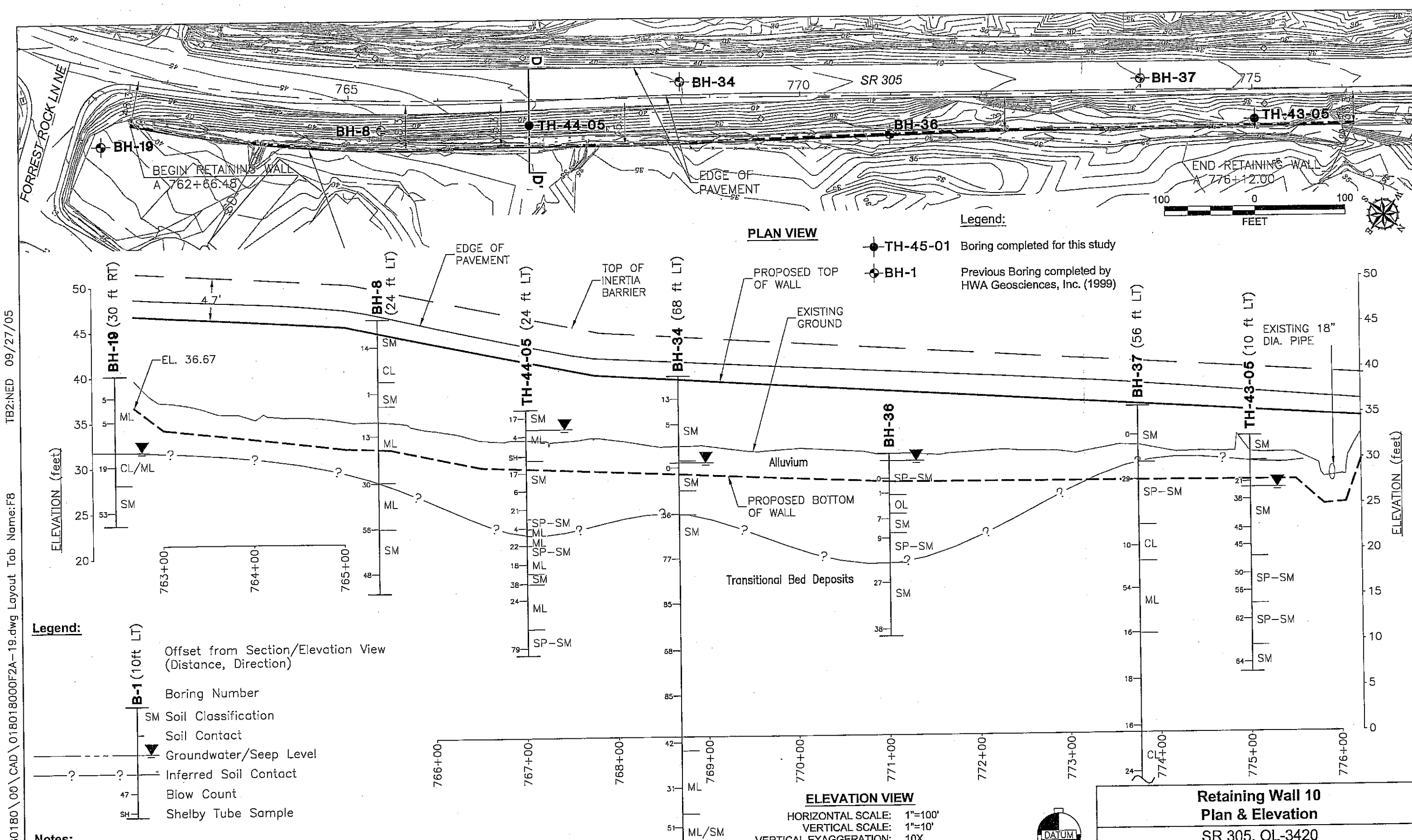

Notes:

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Retaining Wall 10
Plan & Elevation

SR 305, OL-3420
Poulsbo SCL to Bond Road Improvements

GEOENGINEERS

Figure 8

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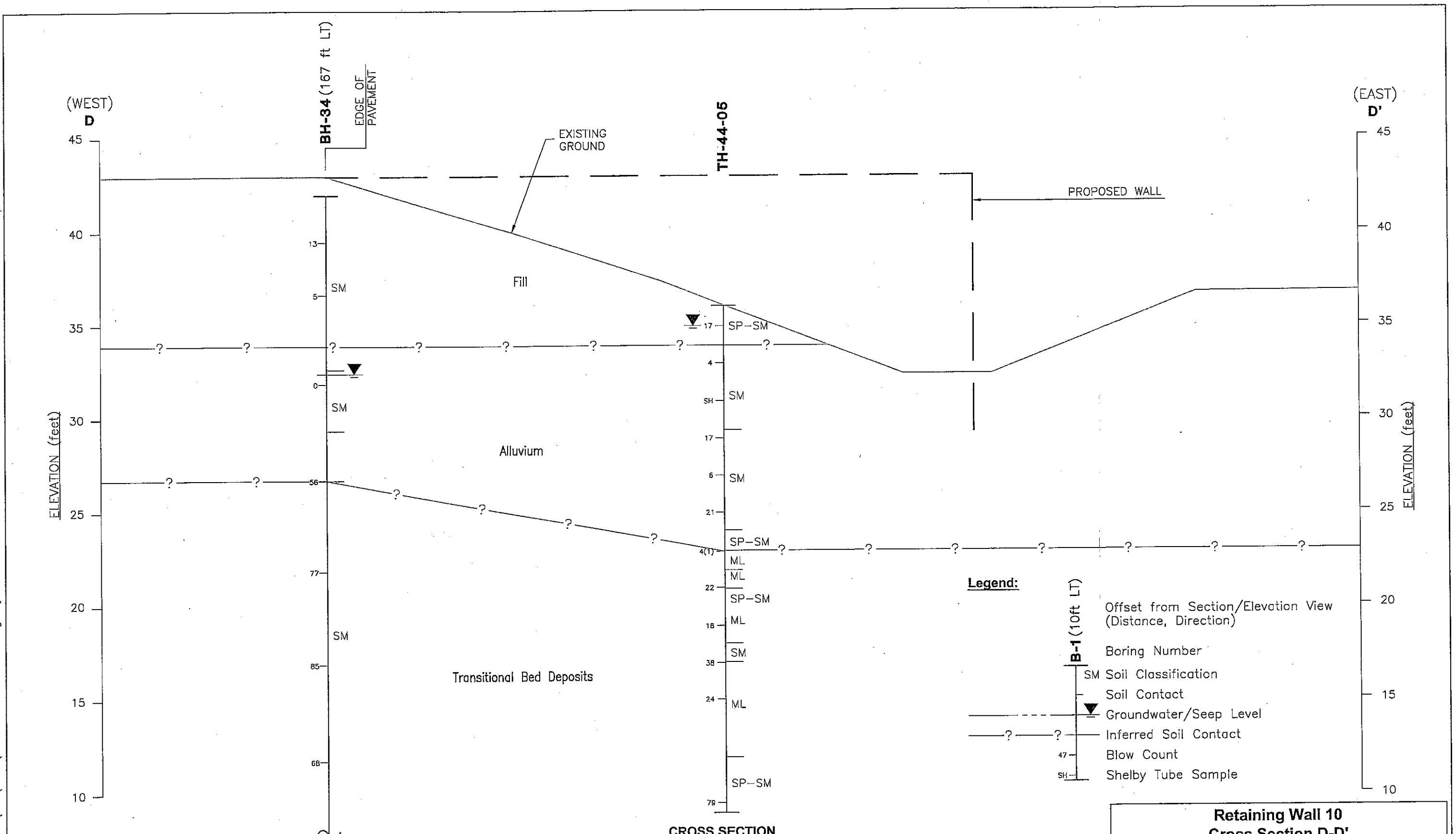

Notes:
1. The subsurface conditions shown are based on interpolation between widely spaced explorations and should be considered approximate; actual subsurface conditions may vary from those shown.
2. Refer to Figure 8 for location of Section D-D'.
3. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record.

Reference: Cross Section developed by GeoEngineers, September 2005.

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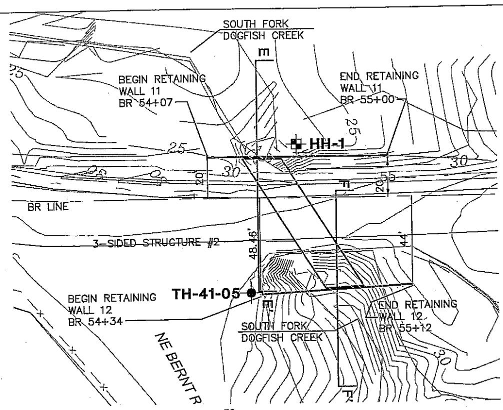
Retaining Wall 10
Cross Section D-D'

SR 305, OL-3420
Poulsbo SCL to Bond Road Improvements

GEOENGINEERS

Figure 9



PLAN VIEW

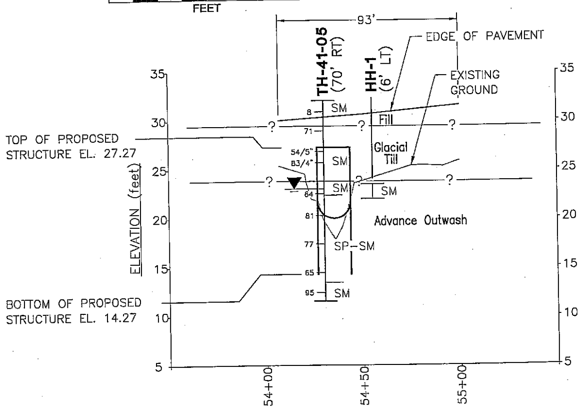
Legend:

- TH-45-01 Boring completed for this study
- HH-1 Hand Exploration completed for this study

DATUM (NAVD) 88

Legend:

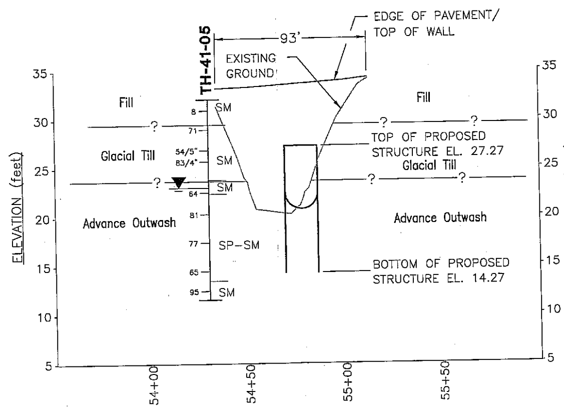
- B-1 (10ft LT) Offset from Section/Elevation View (Distance, Direction)
- Boring Number
- SM Soil Classification
- Soil Contact
- Groundwater/Seep Level
- ? Inferred Soil Contact
- 47 Blow Count
- SH Shelby Tube Sample



RETAINING WALL 11

ELEVATION VIEW

HORIZONTAL SCALE: 1"=50'
VERTICAL SCALE: 1"=10'
VERTICAL EXAGGERATION: 5X



RETAINING WALL 12

ELEVATION VIEW

HORIZONTAL SCALE: 1"=50'
VERTICAL SCALE: 1"=10'
VERTICAL EXAGGERATION: 5X

Notes:

1. The subsurface conditions shown are based on interpolation between widely spaced explorations and should be considered approximate; actual subsurface conditions may vary from those shown.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record.

Reference: Drawing entitled "SR 305, Lincoln Rd. to Bond Rd. - HOV Lanes, Retaining Wall Details" by Washington State Department of Transportation received 06/21/05.

Retaining Wall 11 & 12
Plan & Elevation

SR 305, OL-3420
Poulsbo SCL to Bond Road Improvements

GEOENGINEERS | Figure 10

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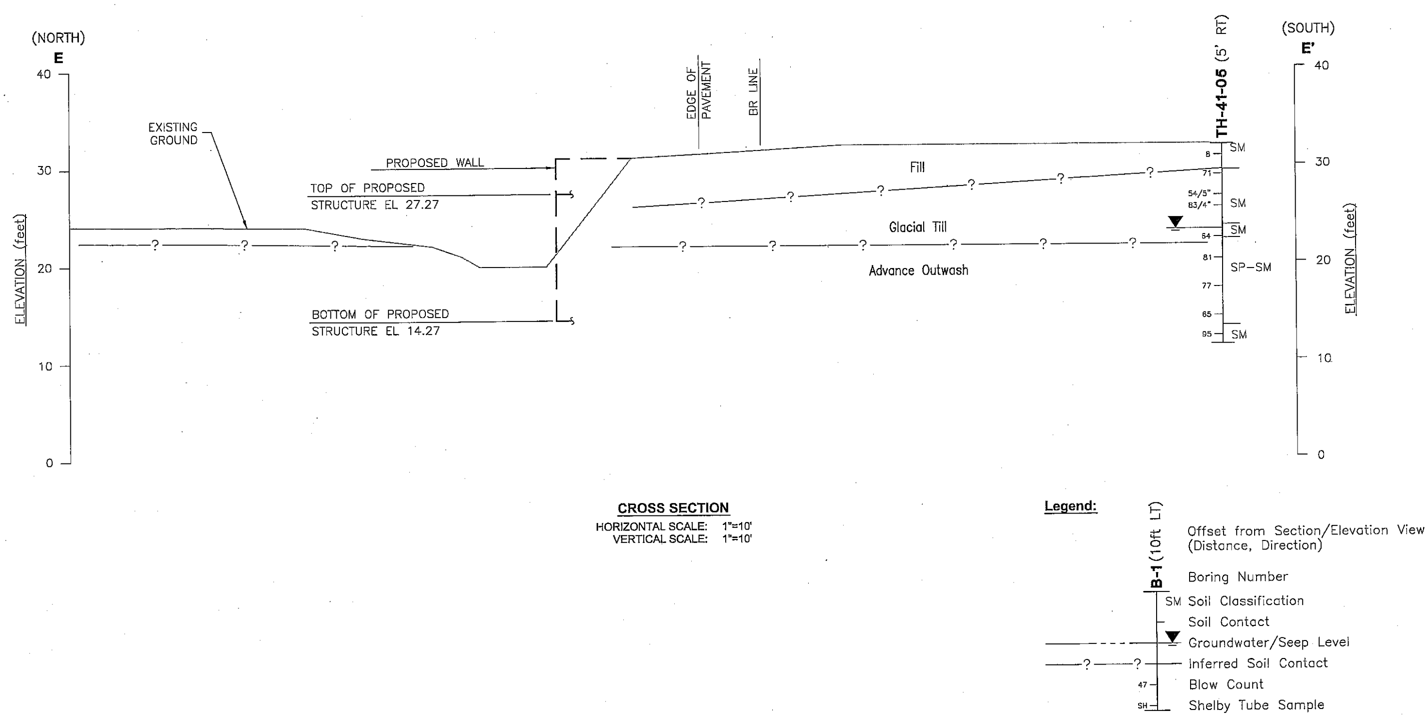

Notes:
1. The subsurface conditions shown are based on interpolation between widely spaced explorations and should be considered approximate; actual subsurface conditions may vary from those shown.
2. Refer to Figure 10 for location of Section E-E'.
3. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record.

Reference: Cross Section developed by GeoEngineers, September 2005.

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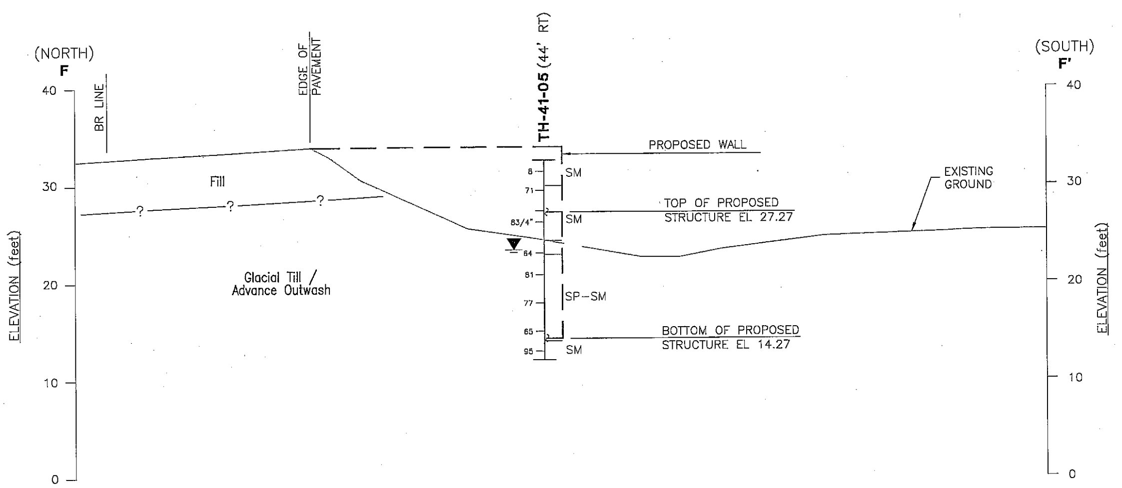
Retaining Wall 11
Cross Section E-E'

SR 305, OL-3420
Poulsbo SCL to Bond Road Improvements

GEOENGINEERS

Figure 11



CROSS SECTION

HORIZONTAL SCALE: 1"=10'
VERTICAL SCALE: 1"=10'

Legend:

B-1 (10ft LT)

- Offset from Section/Elevation View (Distance, Direction)
- Boring Number
- SM Soil Classification
- Soil Contact
- Groundwater/Seep Level
- Inferred Soil Contact
- 47 Blow Count
- SH Shelby Tube Sample

Notes:
1. The subsurface conditions shown are based on interpolation between widely spaced explorations and should be considered approximate; actual subsurface conditions may vary from those shown.
2. Refer to Figure 10 for location of Section F-F'.
3. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record.

Reference: Cross Section developed by GeoEngineers, September 2005.

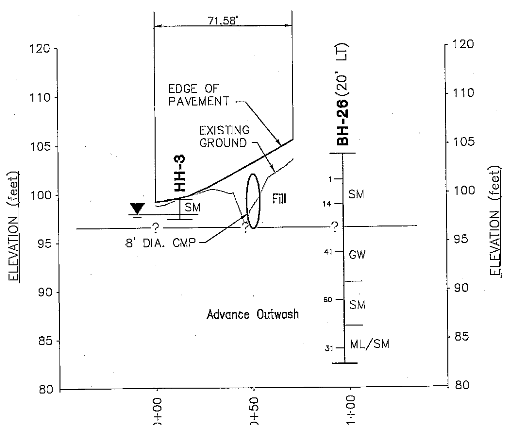
Retaining Wall 12
Cross Section F-F'

SR 305, OL-3420
Poulsbo SCL to Bond Road Improvements

GEOENGINEERS

Figure 12

REDM\P:\ 0180180\00\CAD\0180180000F2A-19.dwg Layout Tab Name:F12 TB2:NED 09/27/05

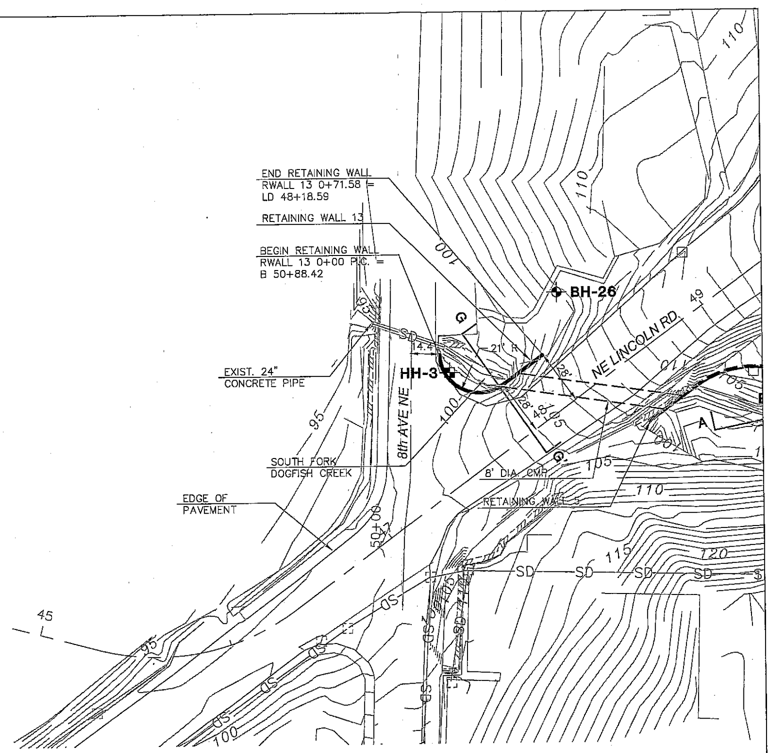


ELEVATION VIEW

HORIZONTAL SCALE: 1"=50'
VERTICAL SCALE: 1"=10'
VERTICAL EXAGGERATION: 5X

Legend:

- B-1 (10ft LT) — Offset from Section/Elevation View (Distance, Direction)
- Boring Number
- SM Soil Classification
- Soil Contact
- Groundwater/Seep Level
- Inferred Soil Contact
- 47 Blow Count
- SH Shelby Tube Sample



PLAN VIEW

50 0 50
FEET

DATUM (NAVD) 88

Legend:

- BH-1 — Previous Boring completed by HWA Geosciences, Inc. (1999)
- HH-1 — Hand Exploration completed for this study

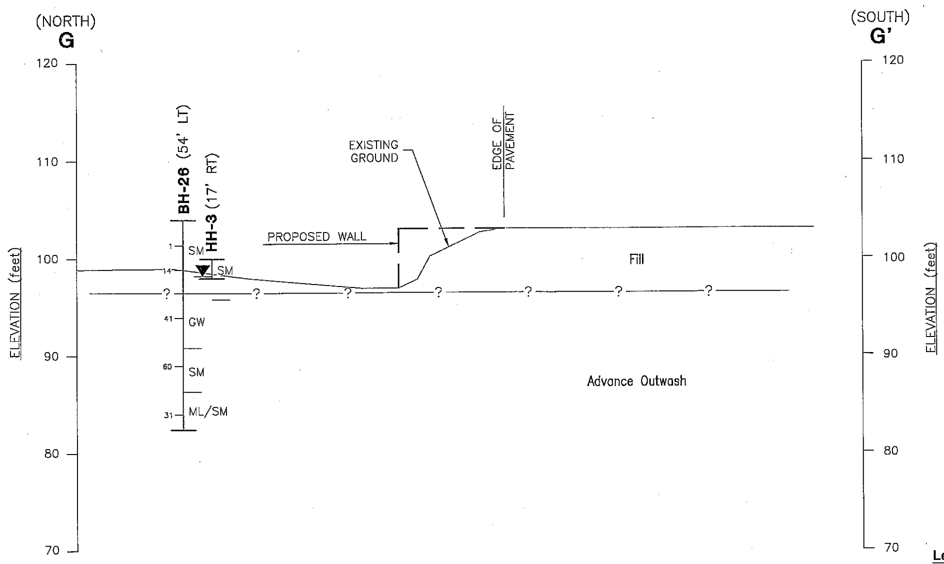
Notes:
1. The subsurface conditions shown are based on interpolation between widely spaced explorations and should be considered approximate; actual subsurface conditions may vary from those shown.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record.

Reference: Drawing entitled "SR 305, Lincoln Rd. to Bond Rd. - HOV Lanes, Retaining Wall Details" by Washington State Department of Transportation received 06/21/05.

Retaining Wall 13
Plan & Elevation
SR 305, OL-3420
Poulsbo SCL to Bond Road Improvements

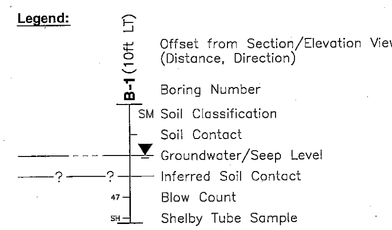
GEOENGINEERS | **Figure 13**

REDM\P:\ 0180180\00\CAD\01801800F2A-19.dwg Layout Tab Name:F13 TB2:NED 09/27/05



CROSS SECTION
HORIZONTAL SCALE: 1"=10'
VERTICAL SCALE: 1"=10'

Legend:



Notes:
1. The subsurface conditions shown are based on interpolation between widely spaced explorations and should be considered approximate; actual subsurface conditions may vary from those shown.
2. Refer to Figure 13 for location of Section G-G'.
3. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record.
Reference: Cross Section developed by GeoEngineers, September 2005.

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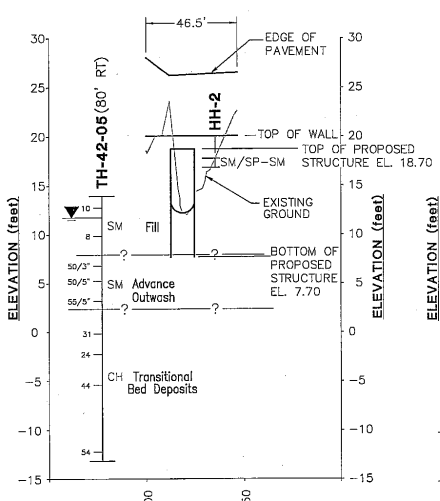
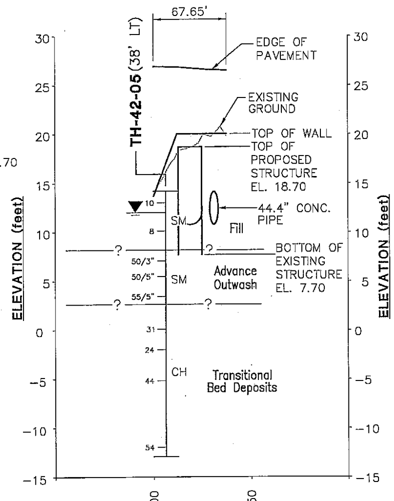
Retaining Wall 13
Cross Section G-G'
SR 305, OL-3420
Poulsbo SCL to Bond Road Improvements
GEOENGINEERS
Figure 14



RETAINING WALL 14

ELEVATION VIEW

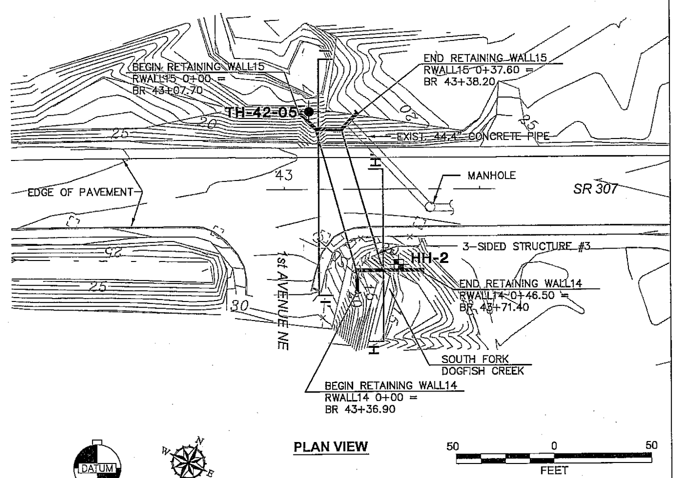
HORIZONTAL SCALE: 1"=50'
VERTICAL SCALE: 1"=10'
VERTICAL EXAGGERATION: 5X



RETAINING WALL 15

ELEVATION VIEW

HORIZONTAL SCALE: 1"=50'
VERTICAL SCALE: 1"=10'
VERTICAL EXAGGERATION: 5X



PLAN VIEW

Legend:

B-1 (10ft LT)
Offset from Section/Elevation View (Distance, Direction)
Boring Number
SM Soil Classification
Soil Contact
Groundwater/Seep Level
Inferred Soil Contact
47 Blow Count
SH Shelby Tube Sample

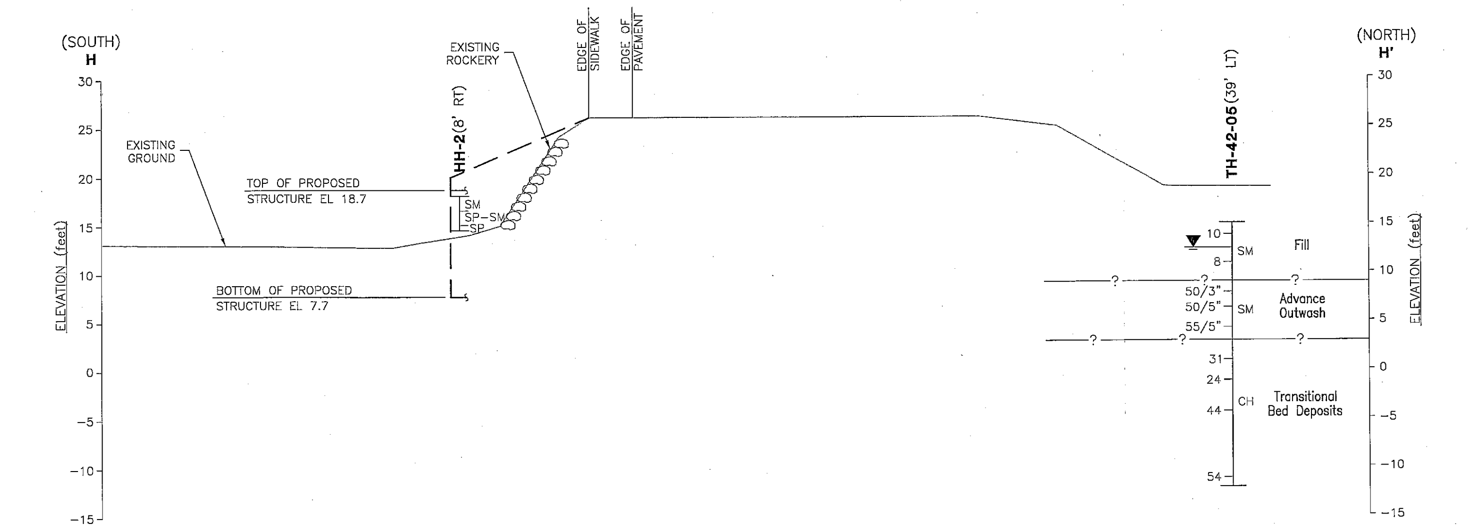
Legend:

TH-45-01 Boring completed for this study
HH-1 Hand Exploration completed for this study

Notes:
1. The subsurface conditions shown are based on interpolation between widely spaced explorations and should be considered approximate; actual subsurface conditions may vary from those shown.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record.
Reference: Drawing entitled "SR 305, Lincoln Rd. to Bond Rd. - HOV Lanes, Retaining Wall Details" by Washington State Department of Transportation received 06/21/05.

Retaining Wall 14 & 15
Plan & Elevation
SR 305, OL-3420
Poulsbo SCL to Bond Road Improvements
GEOENGINEERS
Figure 15

REDM \P:\ 0180180\00\CAD\018018000F2A-19.dwg Layout Tab Name:F15 TB2:NED 09/27/05



CROSS SECTION

HORIZONTAL SCALE: 1"=10'
VERTICAL SCALE: 1"=10'

Legend:

B-1 (10ft LT) — Offset from Section/Elevation View (Distance, Direction)

Boring Number

SM Soil Classification

Soil Contact

Groundwater/Seep Level

Inferred Soil Contact

47 Blow Count

SH Shelby Tube Sample

Notes:

1. The subsurface conditions shown are based on interpolation between widely spaced explorations and should be considered approximate; actual subsurface conditions may vary from those shown.
2. Refer to Figure 15 for location of Section H-H'.
3. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record.

Reference: Cross Section developed by GeoEngineers, September 2005.

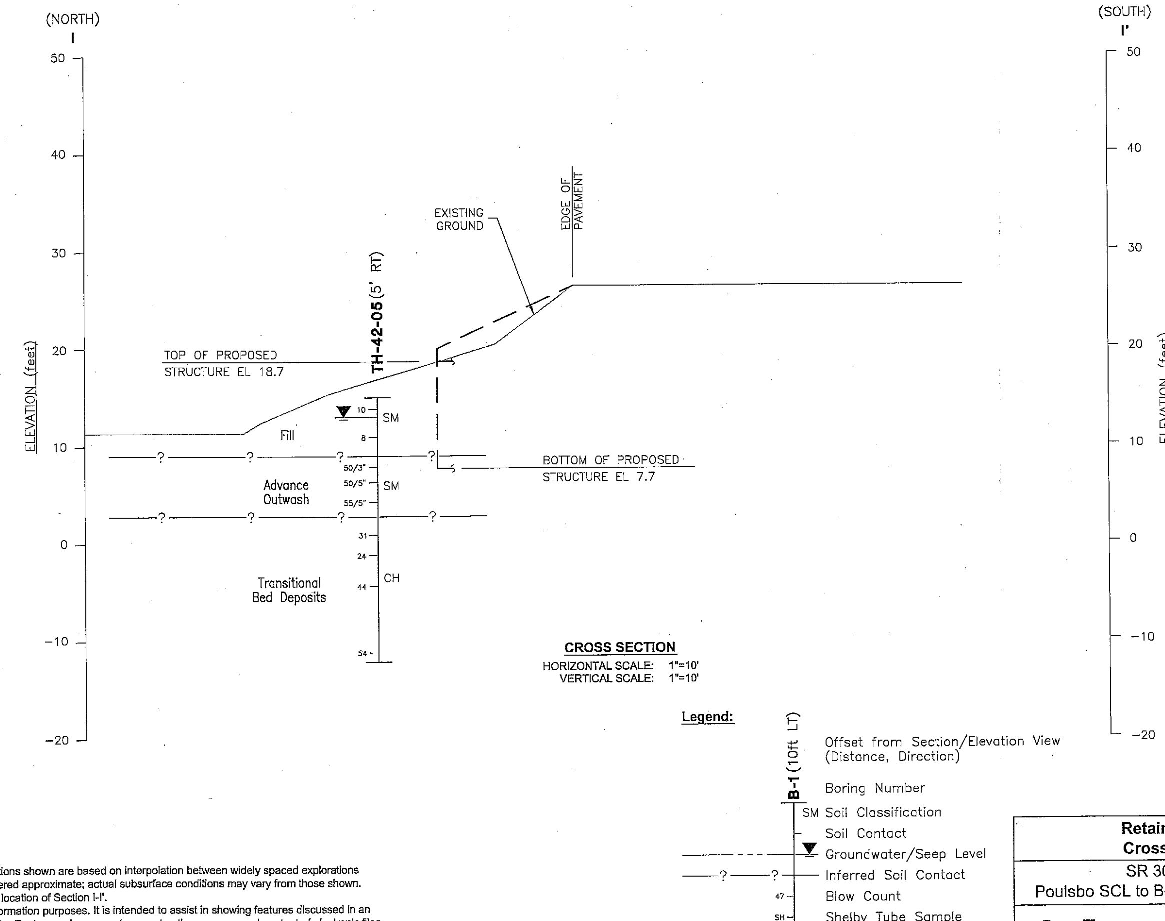
Retaining Wall 14
Cross Section H-H'

SR 305, OL-3420
Poulsbo SCL to Bond Road Improvements

GEOENGINEERS

Figure 16

REDM\P:\0180180\00\CAD\0180180000F2A-19.dwg Layout Tab Name:F16 TB2:NED 09/27/05



Notes:
1. The subsurface conditions shown are based on interpolation between widely spaced explorations and should be considered approximate; actual subsurface conditions may vary from those shown.
2. Refer to Figure 15 for location of Section I-I'.
3. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record.
Reference: Cross Section developed by GeoEngineers, September 2005.

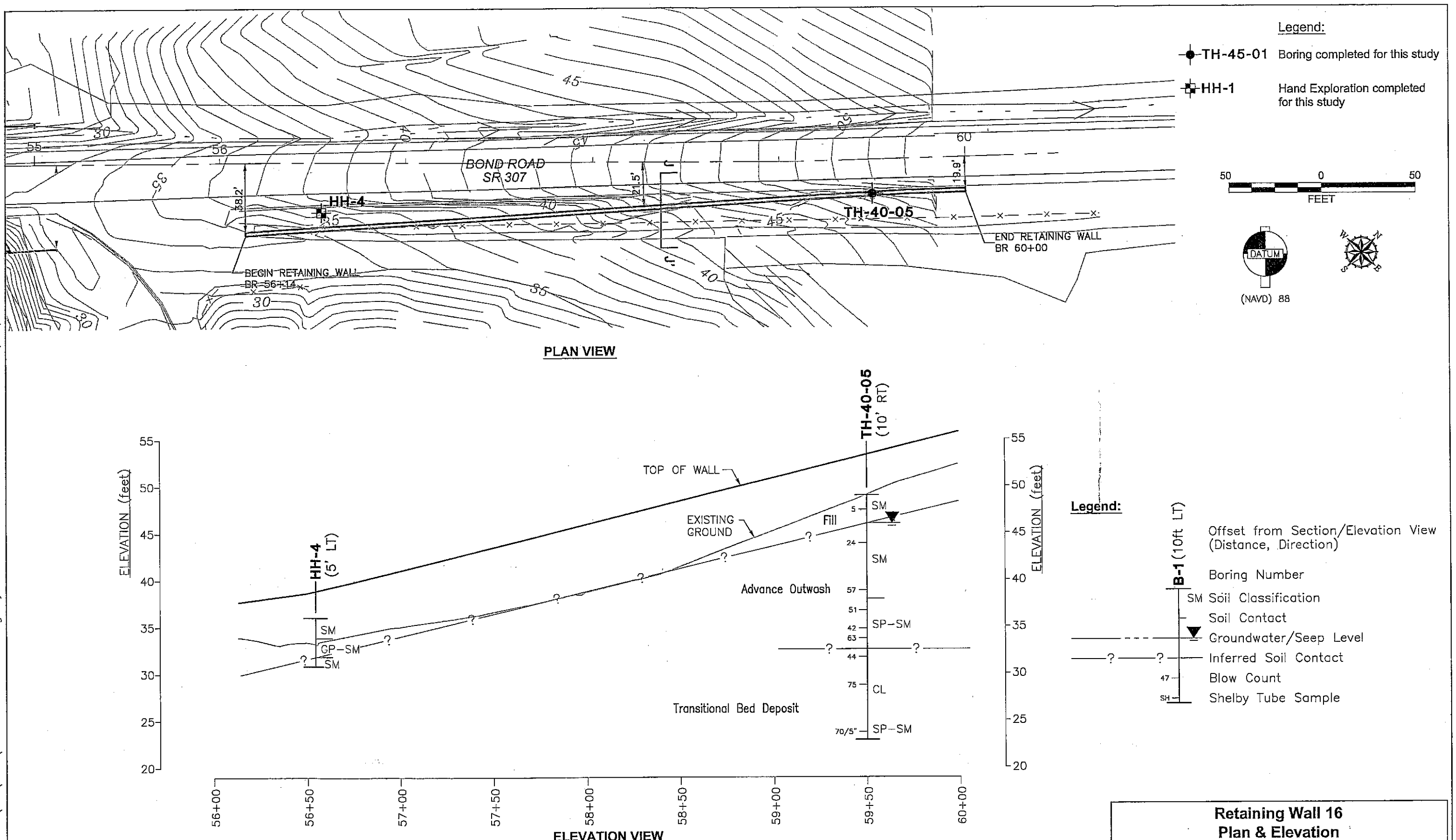
REDM\P:\0180180\00\CAD\018018000F2A-19.dwg Layout Tab Name:F17 TB2:NED 09/27/05

Retaining Wall 15
Cross Section I-I'

SR 305, OL-3420
Poulsbo SCL to Bond Road Improvements

GEOENGINEERS

Figure 17



ELEVATION VIEW

HORIZONTAL SCALE: 1"=50'
VERTICAL SCALE: 1"=10'
VERTICAL EXAGGERATION: 5X

Notes:
1. The subsurface conditions shown are based on interpolation between widely spaced explorations and should be considered approximate; actual subsurface conditions may vary from those shown.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record.

Reference: Drawing entitled "SR 305, Lincoln Rd. to Bond Rd. - HOV Lanes, Retaining Wall Details" by Washington State Department of Transportation received 06/21/05.

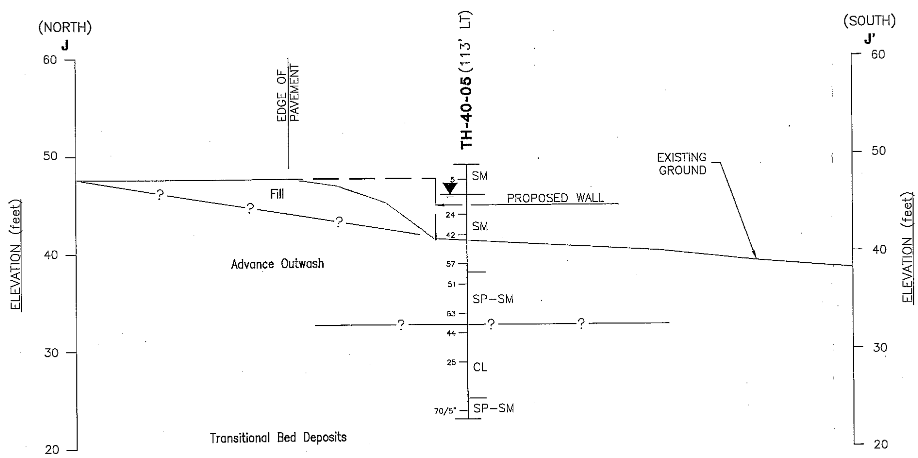
Retaining Wall 16
Plan & Elevation

SR 305, OL-3420
Poulsbo SCL to Bond Road Improvements

GEOENGINEERS

Figure 18

REDM\P:\ 0180180\00\CAD\018018000F2A-19.dwg Layout Tab Name:F18 TB2:NED 09/27/05



CROSS SECTION
HORIZONTAL SCALE: 1"=10'
VERTICAL SCALE: 1"=10'

Legend:

- B-1 (10ft LT) — Offset from Section/Elevation View (Distance, Direction)
- Boring Number
- SM — Soil Classification
- Soil Contact
- Groundwater/Seep Level
- ?—? — Inferred Soil Contact
- 47 — Blow Count
- SH — Shelby Tube Sample

Notes:
1. The subsurface conditions shown are based on interpolation between widely spaced explorations and should be considered approximate; actual subsurface conditions may vary from those shown.
2. Refer to Figure 18 for location of Section J-J'.
3. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record.

Reference: Cross Section developed by GeoEngineers, September 2005.

Retaining Wall 16 Cross-Section J-J'	
SR 305, OL-3420 Poulsbo SCL to Bond Road Improvements	
GEOENGINEERS	**Figure 19**

REDM\P:\ 0180180\00\CAD\01801800F2A-19.dwg Layout Tab Name:F19 TB2:NED 09/27/05

GeoEngineers

Appendix A

Field Explorations And Laboratory Testing Completed For This Study

APPENDIX A
FIELD EXPLORATIONS

GENERAL

Subsurface conditions were explored at the site by drilling seven borings. The explorations were completed to depths ranging from 20½ to 51½ feet below the existing ground surface. The drilling was performed by WSDOT on June 9-12, 2005 and August 1-2, 2005. The locations of the explorations were estimated by measuring distances from site features through taping/pacing in the field, and should be considered approximate. Ground surface elevations at the exploration locations were estimated from contours presented on a topographic map provided by WSDOT. The locations of the explorations are shown on the Site Plan, Figures 2A through 2E.

BORINGS

The borings were completed using cased mud rotary drilling techniques and a CME skid-mounted drill rig. The borings were continuously monitored by a geologist from our firm who examined and classified the soils encountered, obtained representative soil samples, observed groundwater conditions, and prepared a detailed log of each exploration.

The soils encountered in the borings were typically sampled at 2.5- to 5-foot vertical intervals with a 2-inch outside diameter split-barrel standard penetration test (SPT) sampler. The samples were obtained by driving the sampler 18 inches into the soil with a 140-pound hammer free-falling 30 inches. The number of blows required for each 6 inches of penetration is recorded. The blow count ("N-value") of the soil is calculated as the number of blows required for the final 12 inches of penetration. This resistance, or N-value, provides a measure of the relative density of granular soils and the relative consistency of cohesive soils. Where very dense soil conditions preclude driving the full 18-inches, the penetration resistance for the partial penetration is entered on the logs. The blow counts are shown on the boring logs at the respective sample depths.

Relatively undisturbed 3-inch diameter Shelby tube samples were also obtained for laboratory testing.

Soils encountered in the borings were visually classified in general accordance with the classification system described in Figure A-1. A key to the boring log symbols is also presented in Figure A-1. The logs of the borings are presented in Figures A-2 to A-8. The boring logs are based on our interpretation of the field and laboratory data and indicate the various types of soils and groundwater conditions encountered. The logs also indicate the depths at which these soils or their characteristics change; although, the change may actually be gradual. If the change occurred between samples, it was interpreted. The densities noted on the boring logs are based on the blow count data obtained in the borings and judgment based on the conditions encountered.

Observations of groundwater conditions were made during drilling. The groundwater conditions encountered during drilling are presented on the boring logs. Groundwater conditions observed during drilling represent a short term condition and may or may not be representative of the long term groundwater conditions at the site. Groundwater conditions observed during drilling should be considered approximate.

MONITORING WELL INSTALLATION

A representative of GeoEngineers observed the installation of monitoring wells in borings TH-40-05, TH41-05, TH-42-05, TH-44-05 and TH-45-05. The logs for borings where monitoring wells were installed describe the interval over which the monitoring well was screened, depth to groundwater, and the date the groundwater depth was measured. The monitoring wells were constructed using 2-inch diameter PVC (polyvinyl chloride) casing. The depth to which the casing was installed was selected based on our understanding of subsurface soil and groundwater conditions in the project area and the configuration of the proposed facilities in the vicinity of the borehole. The lower portion of the casing was slotted to allow entry of water into the casing. Medium sand was placed in the borehole annulus surrounding the slotted portion of the casing. A bentonite seal was placed above the slotted portion of the casing. The monitoring wells were protected by installing above grade steel monuments set in concrete.

VANE SHEAR TESTS

The in-situ shear strength of the site soils was estimated using a field vane shear device. A vane shear test was performed in boring TH-46-05. The vane shear test consists of inserting a four-bladed vane and rotating it from the surface to estimate the torsional force required to shear a cylindrical surface of the sediment. The resulting torsional force is then converted to a shear surface per unit area.

Peak strengths were evaluated at test intervals of approximately 2 feet. The results of the field vane shear tests are summarized in the following table along with the test depth, the soil type, and the peak shear strengths, as appropriate.

Vane Shear Results from Boring TH-46-05

Vane Shear Designation	Test Depth (feet)	Soil Type	Peak Undrained Shear Strength, (psf)
Test 1	3	Silt	3,250
Test 2	5	Silt	2,225
Test 3	7	Clay	2,075

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	CLEAN GRAVELS		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES
		(LITTLE OR NO FINES)		GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES
	MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	GRAVELS WITH FINES		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
		(APPRECIABLE AMOUNT OF FINES)		GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
MORE THAN 50% RETAINED ON NO. 200 SIEVE	SAND AND SANDY SOILS	CLEAN SANDS		SW	WELL-GRADED SANDS, GRAVELLY SANDS
		(LITTLE OR NO FINES)		SP	POORLY-GRADED SANDS, GRAVELLY SAND
	MORE THAN 50% OF COARSE FRACTION PASSING NO. 4 SIEVE	SANDS WITH FINES		SM	SILTY SANDS, SAND - SILT MIXTURES
		(APPRECIABLE AMOUNT OF FINES)		SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS, ROCK FLOUR, CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
MORE THAN 50% PASSING NO. 200 SIEVE	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS AND SILTS OF MEDIUM TO HIGH PLASTICITY
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: Multiple symbols are used to indicate borderline or dual soil classifications

ADDITIONAL MATERIAL SYMBOLS

SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
	CC	Cement Concrete
	AC	Asphalt Concrete
	CR	Crushed Rock/ Quarry Spalls
	TS	Topsoil/ Forest Duff/Sod

Measured groundwater level in exploration, well, or piezometer

Groundwater observed at time of exploration

Perched water observed at time of exploration

Measured free product in well or piezometer

Stratigraphic Contact

Distinct contact between soil strata or geologic units

Gradual change between soil strata or geologic units

Approximate location of soil strata change within a geologic soil unit

Sampler Symbol Descriptions

2.4-inch I.D. split barrel

Standard Penetration Test (SPT)

Shelby tube

Piston

Direct-Push

Bulk or grab

Blowcount is recorded for driven samplers as the number of blows required to advance sampler 12 inches (or distance noted). See exploration log for hammer weight and drop.

A "P" indicates sampler pushed using the weight of the drill rig.

Laboratory / Field Tests

%F	Percent fines
AL	Atterberg limits
CA	Chemical analysis
CP	Laboratory compaction test
CS	Consolidation test
DS	Direct shear
HA	Hydrometer analysis
MC	Moisture content
MD	Moisture content and dry density
OC	Organic content
PM	Permeability or hydraulic conductivity
PP	Pocket penetrometer
SA	Sieve analysis
TX	Triaxial compression
UC	Unconfined compression
VS	Vane shear

Sheen Classification

NS	No Visible Sheen
SS	Slight Sheen
MS	Moderate Sheen
HS	Heavy Sheen
NT	Not Tested

NOTE: The reader must refer to the discussion in the report text and the logs of explorations for a proper understanding of subsurface conditions. Descriptions on the logs apply only at the specific exploration locations and at the time the explorations were made; they are not warranted to be representative of subsurface conditions at other locations or times.

KEY TO EXPLORATION LOGS

GEOENGINEERS

Figure A-1

Date(s) Drilled	06/09/05 - 06/10/05	Logged By	MR4	Checked By	KGO
Drilling Contractor	WSDOT	Drilling Method	Mud Rotary	Sampling Methods	SPT, Shelby Tube
Auger Data	HW 4-inch ID	Hammer Data	140 lb hammer/ in drop autohammer	Drilling Equipment	CME-45 Skid Rig
Total Exploration Depth (ft)	26	Ground Surface Elevation (ft)	Approximately 42	Groundwater Level (ft. bgs)	5.5
Vertical Datum		Datum/ System		Easting(x): Northing(y):	

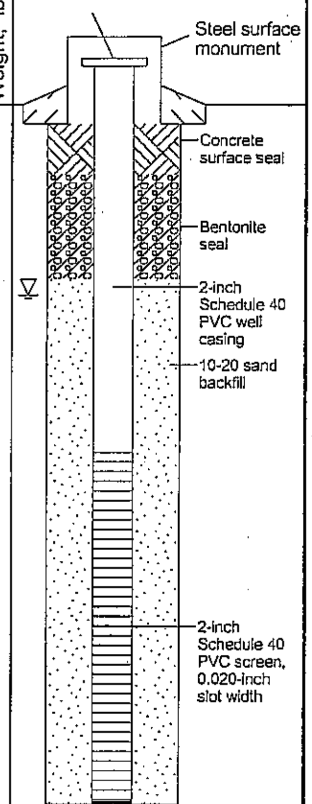
Elevation feet	Depth feet	Recovered (in)	Blows/foot	Sample Number	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	Dry Unit Weight, lbs/ft³
	0			1	SM	Brown silty fine to medium sand with organic matter (loose, moist to wet) (fill)		
			5	2			40	
		18	24	3	SM	Brownish gray silty fine to medium sand with gravel (dense to very dense, wet) (advance outwash)		
	5	18	42	4		Occasional thin silt lense (SA)	12	
		16	57	5				
	10	18	51	6	SP-SM	Gray fine to medium sand with silt and occasional gravel (very dense, moist)		
		14	63	7	SP-SM	Gray fine to coarse sand with silt and gravel (very dense, wet)		
	15	12	44	8	CL	Gray clay (hard, moist) (transitional bed deposits)		
			25	9		(AL)	29	
	20							
		12	70/5"	10	SP-SM	Gray fine to medium sand with silt (very dense, wet)		
	25							

Groundwater measured at 5.5 feet below the ground surface on 09/09/05

30

35

WELL CONSTRUCTION



Note: See Figure A-1 for explanation of symbols.

V6_GTWELL P:\0\0180180\00\FINALS\018018000.GPJ GEIV6_1.GDT 9/27/05

LOG OF MONITORING WELL TH-40-05

GeoEngineers

Project: SR 305 OL-3420 SCL to Bond Road
Project Location: Poulsbo, Washington
Project Number: 0180-180-00

Figure A-2
Sheet 1 of 1

Date(s) Drilled	06/10/05	Logged By	MR4	Checked By	KGO
Drilling Contractor	WSDOT	Drilling Method	Mud Rotary	Sampling Methods	SPT, Shelby Tube
Auger Data	HW 4-inch ID	Hammer Data	140 lb hammer/ in drop autohammer	Drilling Equipment	CME-45 Skid Rig
Total Exploration Depth (ft)	20.5	Ground Surface Elevation (ft)	Approximately 32	Groundwater Level (ft. bgs)	9
Vertical Datum		Datum/ System		Easting(x): Northing(y):	

Elevation feet	Depth feet	SAMPLES: Recovered (in)	Blows/foot	Sub-Sample Sample Number	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	Dry Unit Weight, lbs/ft³
	0	15	8	1	SM	Brown silty fine to medium sand with occasional gravel and organic matter (loose, moist) (fill)	16	
		14	71	2			16	
					SM	Gray silty fine to medium sand with gravel (very dense, moist) (glacial till)		
		12	54/5"	3			6	
	5					(SA)		
		6	83/4"	4			9	
		18	64	5	SM	Gray silty fine sand with silt lenses (very dense, wet) (advance outwash)		
	10				SP	Gray fine sand (very dense, wet)		
		18	81	6				
		18	77	7				
	15							
		18	65	8		Occasional lenses of silty fine sand		
		14	95	9	SM	Gray silty fine to coarse sand (very dense, wet)		
	20							

Groundwater measured at 9.1 feet below the ground surface 08/02/05

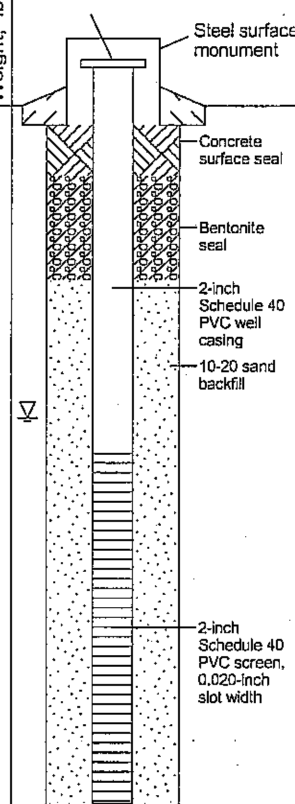
Groundwater measured at 9.3 feet below the ground surface on 09/09/05

25

30

35

WELL CONSTRUCTION



Note: See Figure A-1 for explanation of symbols.

LOG OF MONITORING WELL TH-41-05

GeoEngineers

Project: SR 305 OL-3420 SCL to Bond Road

Project Location: Poulsbo, Washington

Project Number: 0180-180-00

Figure A-3

Sheet 1 of 1

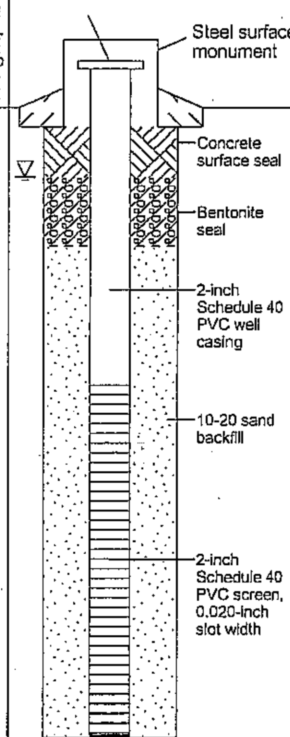
Date(s) Drilled	06/12/05	Logged By	MR4	Checked By	KGO
Drilling Contractor	WSDOT	Drilling Method	Mud Rotary	Sampling Methods	SPT, Shelby Tube
Auger Data	HW 4-inch ID	Hammer Data	140 lb hammer/ in drop autohammer	Drilling Equipment	CME-45 Skid Rig
Total Exploration Depth (ft)	27	Ground Surface Elevation (ft)	Approximately 15	Groundwater Level (ft. bgs)	2
Vertical Datum		Datum/ System		Easting(x): Northing(y):	

Elevation feet	Depth feet	SAMPLES: Recovered (in)	Blows/foot	Sample Number	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	Dry Unit Weight, lbs/ft³
	0	18	10	1	SM	Brown silty fine to medium sand with occasional gravel and organic matter (loose, moist) (roots) (topsoil)	16	
					ML	Brown silt with sand (stiff, moist) (oxidation staining) (fill)		
		10	8	2	SM	Brown silty fine to medium sand with gravel and wood organics (loose, wet)		
	5							
		10	50/3"	3	SM	Gray silty fine sand (very dense, moist to wet) (advance outwash)		
		11	50/5"	4		Grades to fine to medium silty sand with occasional gravel		
	10	12	55/5"	5				
					CH	Gray fat clay (very stiff to hard, moist) (transitional bed deposits)		
		22	31	6				
	15	24	24	7		AL	30	
		24	44	8				
	20					(Slickensides)		
	25	22	54	9		(Slickensides)		

Groundwater measured at 2.2 feet below the ground surface 08/02/05

Groundwater measured at 2.1 feet below the ground surface on 09/09/05

WELL CONSTRUCTION



Note: See Figure A-1 for explanation of symbols.

V6_GTWELL P:\0\0180180\00\FINALS\018018000.GPJ GEIV6_1.GDT 9/27/05

LOG OF MONITORING WELL TH-42-05

GEOENGINEERS

Project: SR 305 OL-3420 SCL to Bond Road

Project Location: Poulsbo, Washington

Project Number: 0180-180-00

Figure A-4

Sheet 1 of 1

Date(s) Drilled	06/11/05	Logged By	MR4	Checked By	KGO
Drilling Contractor	WSDOT	Drilling Method	Mud Rotary	Sampling Methods	SPT, Shelby Tube
Auger Data	HW 4-inch ID	Hammer Data	140 lb hammer/ in drop autohammer	Drilling Equipment	CME-45 Skid Rig
Total Depth (ft)	26	Surface Elevation (ft)	Approximately 33	Groundwater Level (ft. bgs)	6
Vertical Datum		Datum/ System		Easting(x): Northing(y):	

Elevation feet	Depth feet	SAMPLES Interval	Recovered (in)	Blows/foot	Sub-Sample	Sample Number	Water Level	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	Dry Unit Weight, lbs/ft³	OTHER TESTS AND NOTES
	0					1			SM	Brown fine to coarse sand with silt, gravel and organic matter (loose, moist) (alluvium)	3		
									SM	Brownish gray silty fine sand (medium dense to dense, wet) (transitional bed deposits)			
			18	21		2					20		
	5												
			20	38		3	∇				17		SA
			20	45		4							
	10												
			24	45		5							
									SP-SM	Brownish gray fine to medium sand with silt (very dense, wet)			
			20	50		6							
	15												
				56		7							
									SP-SM	Brownish gray fine to coarse sand with silt (very dense, wet)			
				62		8							
	20												
									SM	Brown silty fine to coarse sand with gravel (very dense, wet)			
			18	64		9							
	25												
	30												
	35												

Note: See Figure A-1 for explanation of symbols.

V6 GTBORING P:\0\0180180\00\FINALS\018018000.GPJ GEIV6_1.GDT 9/27/05

LOG OF BORING TH-43-05

GeoEngineers

Project: SR 305 OL-3420 SCL to Bond Road
Project Location: Poulsbo, Washington
Project Number: 0180-180-00

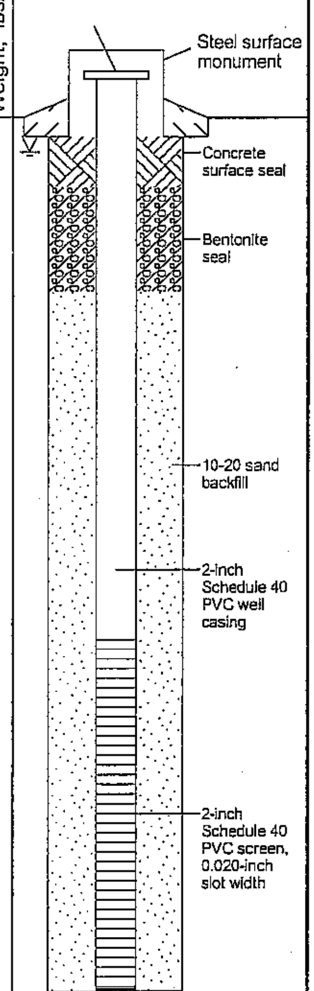
Figure A-5
Sheet 1 of 1

Date(s) Drilled	06/11/05	Logged By	Tybarra (WSDOT)	Checked By	KGO
Drilling Contractor	WSDOT	Drilling Method	Mud Rotary	Sampling Methods	SPT
Auger Data	HW 4-inch ID	Hammer Data	140 lb hammer/ in drop autohammer	Drilling Equipment	CME-45 Skid Rig
Total Exploration Depth (ft)	27	Ground Surface Elevation (ft)	Approximately 36	Groundwater Level (ft. bgs)	1
Vertical Datum		Datum/ System		Easting(x): Northing(y):	

Elevation feet	Depth feet	SAMPLES: Interval	Recovered (in)	Blows/foot	Sub-Sample Sample Number	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	Dry Unit Weight, lbs/ft^3
	0		18	17	1		SP-SM	Brownish gray fine to medium sand with silt and occasional gravel (medium dense, moist) (fill?)	7	
			18	4	2		SM	Brown silty fine to medium sand with occasional gravel and organic matter (alluvium)	20	
			4	P	3		ML	Brown to gray interlayered silt with sand to sandy silt with organic matter (soft, wet)		
	5		24	17	4					
							SM	Gray silty fine sand (loose to medium dense, wet)		
			24	6	5			(SA)	24	
	10		19	21	6					
			24	4	7		SM/ML	Gray interlayered silty fine sand and gray silt (loose/soft, wet)		
			24	22	8		SP-SM	Gray fine sand with silt (medium dense, wet) (transitional bed deposits)		
	15		22	18	9		ML	Gray silt/sandy silt (very stiff, moist)		
			20	38	10		SM/ML	Gray interlayered silty fine sand and silt (dense/hard, moist to wet)		
	20		22	24	11		ML	Gray silt (very stiff, moist)		
							SP-SM	Gray fine sand with silt (very dense, wet)		
	25		12	79	12					
	30									
	35									

Groundwater measured at 2.2 feet below the ground surface on 08/02/05
Groundwater measured at 1.2 feet below the ground surface on 09/09/05

WELL CONSTRUCTION



Note: See Figure A-1 for explanation of symbols.

LOG OF MONITORING WELL TH-44-05

GEOENGINEERS

Project: SR 305 OL-3420 SCL to Bond Road
Project Location: Poulsbo, Washington
Project Number: 0180-180-00

Figure A-6
Sheet 1 of 1

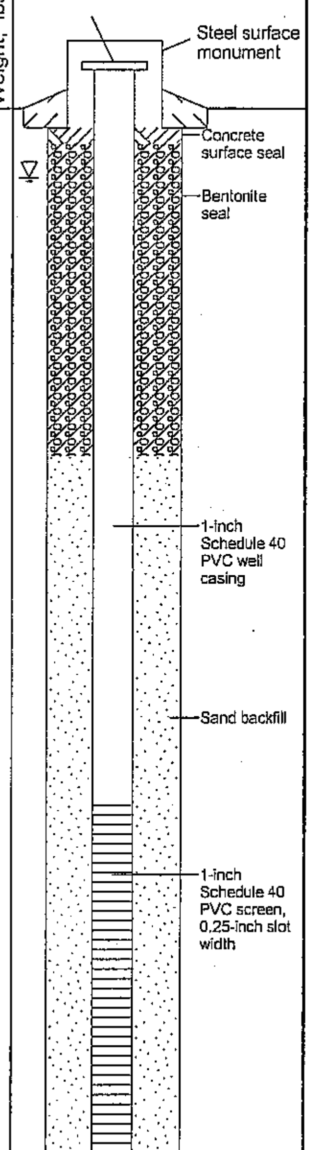
V6_GTWELL P:\0\0180180\00\FINALS\018018000.GPJ GEIV6_1.GDT 9/27/05

Date(s) Drilled	08/02/05	Logged By	ABA	Checked By	KGO
Drilling Contractor	WSDOT	Drilling Method	Mud Rotary	Sampling Methods	SPT, Shelby Tube
Auger Data	3.4-inch ID	Hammer Data	140 lb hammer/30 in drop autohammer	Drilling Equipment	CME-45 Skid Rig
Total Exploration Depth (ft)	30.5	Ground Surface Elevation (ft)	Approximately 40	Groundwater Level (ft. bgs)	2
Vertical Datum		Datum/ System		Easting(x): Northing(y):	

Elevation feet	Depth feet	SAMPLES: Recovered (in)	Blows/foot	Sample Number	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	Dry Unit Weight, lbs/ft³
	0	2	14	1	GP	Brown fine to coarse gravel with silt and sand and wood debris (medium dense, moist to wet) (fill)	14	
		6	24	2				
					WD	(Drilled through 1 foot of wood)		
		10	38	3	SM	Gray silty fine to medium sand with gravel (dense, wet) (transitional bed deposits)	13	
	5							
		12	36	4	SP	Gray fine to medium sand with gravel and a trace silt (dense, wet) (SA)	12	
		14	42	5				
	10				SP-SM	Gray fine to medium sand with silt and interbedded layers of sandy silt (dense, wet)		
		16	52	6		Grades to very dense	17	
		18	78	7				
	15							
		17	99	8				
	20							
		15	80	9				
	25							
		18	70/10"	10				
	30							
	35							

Groundwater measured at 2.1 feet below the ground surface on 09/09/05

WELL CONSTRUCTION



Note: See Figure A-1 for explanation of symbols.

LOG OF MONITORING WELL TH-45-05

GEOENGINEERS

Project: SR 305 OL-3420 SCL to Bond Road
Project Location: Poulsbo, Washington
Project Number: 0180-180-00

Figure A-7
Sheet 1 of 1

Date(s) Drilled	08/01/05	Logged By	ABA	Checked By	KGO
Drilling Contractor	WSDOT	Drilling Method	Mud Rotary	Sampling Methods	SPT, Shelby Tube
Auger Data	3.4-inch ID	Hammer Data	140 lb hammer/30 in drop autohammer	Drilling Equipment	CME-45 Skid Rig
Total Depth (ft)	51.5	Surface Elevation (ft)	Approximately 48	Groundwater Level (ft. bgs)	15
Vertical Datum		Datum/ System		Easting(x): Northing(y):	

Elevation feet	Depth feet	SAMPLES: Interval	Recovered (in)	Blows/foot	Sub-Sample Sample Number	Water Level	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	Dry Unit Weight, lbs/ft³	OTHER TESTS AND NOTES
	0		6	10	1			ML	Dark brown silt with sand, charcoal and organic matter (medium stiff, moist) (alluvium)			
			14	8	2			ML	Gray silt with occasional sand and roots, organic matter (medium stiff, moist)	32		
	5		4	5	3					28		
			18	1	4			CL	Gray clay interbedded with silty fine sand (very soft, moist to wet)	48		
	10		18	P	5					29	97	AL, TX, CS
			18	22	6					38		
								ML	Gray silt with interbedded sandy silt and silty fine sand (very stiff, moist to wet)			
	15					∇		SP-SM	Gray fine sand with silt (medium dense, wet)			
			18	10	7					39		
								ML	Gray silt (soft, wet)			
			18	3	8							
	20		12	P	9					34	81	AL, CS, TX
								SM	Gray silty fine sand (very dense, wet) (transitional bed deposits)			
	25		18	57	10							
	30								Grades to silty fine to medium sand (medium dense, wet)			
			18	25	11							
								CH	Gray fat clay (very stiff, moist)			
	35											

Note: See Figure A-1 for explanation of symbols.

V6_GTBORING P:\0\0180180\00\FINALS\018018000.GPJ GEIV6_1.GDT 9/27/05

LOG OF BORING TH-46-05

GEOENGINEERS

Project: SR 305 OL-3420 SCL to Bond Road
Project Location: Poulsbo, Washington
Project Number: 0180-180-00

Figure A-8
Sheet 1 of 2

Elevation feet	Depth feet	SAMPLES: Interval	SAMPLES: Recovered (in)	SAMPLES: Blows/foot	SAMPLES: Sub-Sample Sample Number	Water Level	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	Dry Unit Weight, lbs/ft^3	OTHER TESTS AND NOTES
	35		18	16	12					34		
	40		18	19	13					26		AL
	45		18	17	14							
	50		18	17	15							
	55											
	60											
	65											
	70											
	75											

V6_GTBORING P:\0\0180180\00\FINALS\018018000.GPJ GEIV6_1.GDT 9/27/05

LOG OF BORING TH-46-05 (continued)

GeoEngineers

Project: SR 305 OL-3420 SCL to Bond Road
Project Location: Poulsbo, Washington
Project Number: 0180-180-00

Figure A-8
Sheet 2 of 2

Date Excavated: 06/12/05

Logged by: MR4

Equipment: Hand Auger

Surface Elevation (ft): Approximately 23

Elevation feet	Depth feet	Sample	Sample Number	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	OTHER TESTS AND NOTES
	0				SP-SM	Brown fine to coarse sand with silt (loose to medium dense, moist) (weathered advance outwash)		
					SM	Brown silty fine to medium sand with gravel (loose, moist)		

Refusal at 1.5 feet on root
Hand auger completed at 1.5 feet on 06/12/05
No groundwater seepage observed
No caving observed

5

10

Notes: See Figure A-1 for explanation of symbols.
The depths on the hand auger logs are based on an average of measurements across the hand auger and should be considered accurate to 0.5 foot.

LOG OF HAND AUGER HH-1

GEOENGINEERS

Project: SR 305 OL-3420 SCL to Bond Road
Project Location: Poulsbo, Washington
Project Number: 0180-180-00

Figure A-9
Sheet 1 of 1

V6_GTTPIT P:\0\0180180\00\FINALS\018018000.GPJ GEIV6_1.GDT 9/27/05

Date Excavated: 06/13/05

Logged by: MR4

Equipment: Hand Auger

Surface Elevation (ft): Approximately 17

Elevation feet	Depth feet	Sample	Sample Number	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	OTHER TESTS AND NOTES
	0				SM	Brown silty fine sand with organic matter (topsoil)		
		▼			SP-SM	Grayish brown fine sand with silt and organic matter (medium dense, wet)		
					SP	Gray fine to coarse sand with gravel and organic matter (medium dense to dense, wet)		

Hand auger completed at 3.5 feet on 06/13/05
Groundwater seepage observed at approximately 2 feet
No caving observed

5

10

Notes: See Figure A-1 for explanation of symbols.
The depths on the hand auger logs are based on an average of measurements across the hand auger and should be considered accurate to 0.5 foot.

LOG OF HAND AUGER HH-2

GEOENGINEERS

Project: SR 305 OL-3420 SCL to Bond Road
Project Location: Poulsbo, Washington
Project Number: 0180-180-00

Figure A-10
Sheet 1 of 1

Date Excavated:	06/13/05	Logged by:	MR4
Equipment:	Hand Auger	Surface Elevation (ft):	Approximately 100

Elevation feet	Depth feet	Sample	Sample Number	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	OTHER TESTS AND NOTES
	0				SM	Brown fine to coarse sand with gravel and organic matter (loose, moist) (fill)		
						Grades to wet		
					SM	Brownish gray silty fine sand with organic matter (medium dense, wet)		
		▼			SM	Grayish brown silty fine to coarse sand with gravel (dense, wet) (advance outwash)		

Hand auger completed at 2.2 feet on 06/13/05
Groundwater seepage observed at 1.4 feet
No caving observed

5

10

Notes: See Figure A-1 for explanation of symbols.
The depths on the hand auger logs are based on an average of measurements across the hand auger and should be considered accurate to 0.5 foot.

LOG OF HAND AUGER HH-3

GEOENGINEERS

Project: SR 305 OL-3420 SCL to Bond Road
Project Location: Poulsbo, Washington
Project Number: 0180-180-00

Figure A-11
Sheet 1 of 1

Date Excavated: 08/02/05 | Logged by: ABA
Equipment: Hand Auger | Surface Elevation (ft): Approximately 36

Elevation feet	Depth feet	Sample	Sample Number	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	OTHER TESTS AND NOTES
	0				XX	Gray crushed shells with sand and organic matter (fill)		
		X	1		SM	Brown silty fine sand with gravel and shell fragments (medium dense, moist) (fill)		
		X	2		GP-GM	Brown fine to coarse gravel with silt, sand and occasional cobbles (crushed concrete) (dense, moist)		
		X	3					
		X	4		SM	Gray-brown silty fine sand with gravel and roots (medium dense, moist) (advance outwash)		
	5					Hand auger completed at 5 feet on 08/02/05 No groundwater seepage observed No caving observed		
	10							

Notes: See Figure A-1 for explanation of symbols.
The depths on the hand auger logs are based on an average of measurements across the hand auger and should be considered accurate to 0.5 foot.

LOG OF HAND AUGER HH-4

GeoEngineers

Project: SR 305 OL-3420 SCL to Bond Road
Project Location: Poulsbo, Washington
Project Number: 0180-180-00

Figure A-12
Sheet 1 of 1

GEOENGINEERS

APPENDIX B
LABORATORY TESTING

APPENDIX B
LABORATORY TESTING

GENERAL

Soil samples obtained from the explorations were transported to our laboratory and examined to confirm or modify field classifications, as well as to evaluate index properties of the soil samples. Representative samples were selected for laboratory testing consisting of the determination of the moisture content, percent fines, grain size distribution (sieve analyses), and Atterberg limits (plasticity characteristics). Representative undisturbed samples were also selected for laboratory testing consisting of consolidation and triaxial testing. The tests were performed in general accordance with test methods of the American Society for Testing and Materials (ASTM) or other applicable procedures.

The sieve analysis and Atterberg limits test results are presented in Figures B-1 through B-4. The consolidation and triaxial testing results are presented in Figures B-5 through B-8. The results of the moisture content determinations are presented at the respective sample depth on the exploration logs in Appendix A.

MOISTURE CONTENT TESTING

Moisture content tests were completed in general accordance with ASTM D 2216 for representative samples obtained from the explorations. The results of these tests are presented on the exploration logs in Appendix A at the depths at which the samples were obtained.

SIEVE ANALYSES

Sieve analyses were performed on selected samples in general accordance with ASTM D 422 to determine the sample grain size distribution. The wet sieve analysis method was used to determine the percentage of soil greater than the U.S. No. 200 mesh sieve. The results of the sieve analyses were plotted, classified in general accordance with the Unified Soil Classification System (USCS), and are presented in Figures B-1 and B-2.

ATTERBERG LIMITS TESTING

Atterberg limits tests were performed on selected fine-grained soil samples. The tests were used to classify the soil as well as to evaluate index properties. The liquid limit and the plastic limit were estimated through a procedure performed in general accordance with ASTM D 4318. The results of the Atterberg limits tests are summarized in Figures B-3 and B-4.

CONSOLIDATION TESTING

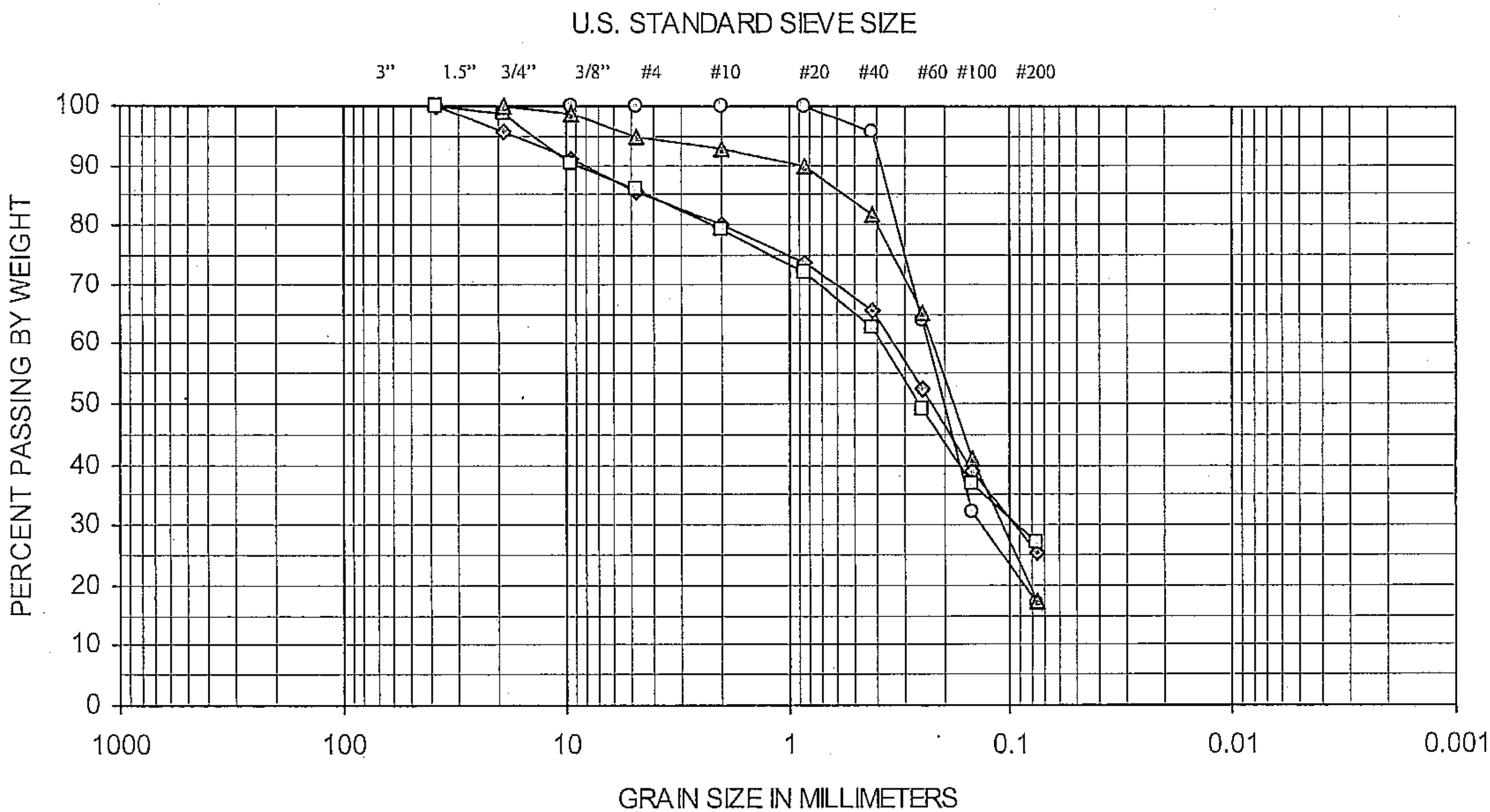
One-dimensional consolidation tests were performed on selected fine-grained soil samples. The tests were used to determine time-rate of consolidation and consolidation parameters for engineering analysis. The consolidation tests were performed in general accordance with ASTM D 2435. The results of the consolidation tests are summarized in Figures B-5 and B-6.

TRIAXIAL TESTING

Consolidated-undrained (CU) triaxial tests with pore water pressure measurements were performed on selected undisturbed soil samples. The triaxial tests were used to determine soil strength parameters for engineering analysis. The tests were completed at confining pressures ranging from about 1300 psf to 4000 psf to model both the current pressure state as well as the confining pressures anticipated after placement of additional fill.

The CU triaxial compression tests were completed in general accordance with ASTM D 4767. The applied vertical load, pore water pressure, and strain were recorded electronically during the test. The raw data were corrected to account for the horizontal resistance of the rubber membrane. The results of the CU triaxial compression tests are presented in Figures B-7 and B-8.

0180-180-00 KGO : HMS : jvj 6-29-05 (Sieve.ppt)



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

SYMBOL	EXPLORATION NUMBER	DEPTH (ft)	SOIL CLASSIFICATION
◈	TH-40-05	7	Brownish gray silty fine to medium sand with gravel (SM)
□	TH-41-05	5	Gray silty fine to medium sand with gravel (SM)
◎	TH-43-05	7	Brownish gray silty fine sand (SM)
△	TH-44-05	9	Gray silty fine sand (SM)

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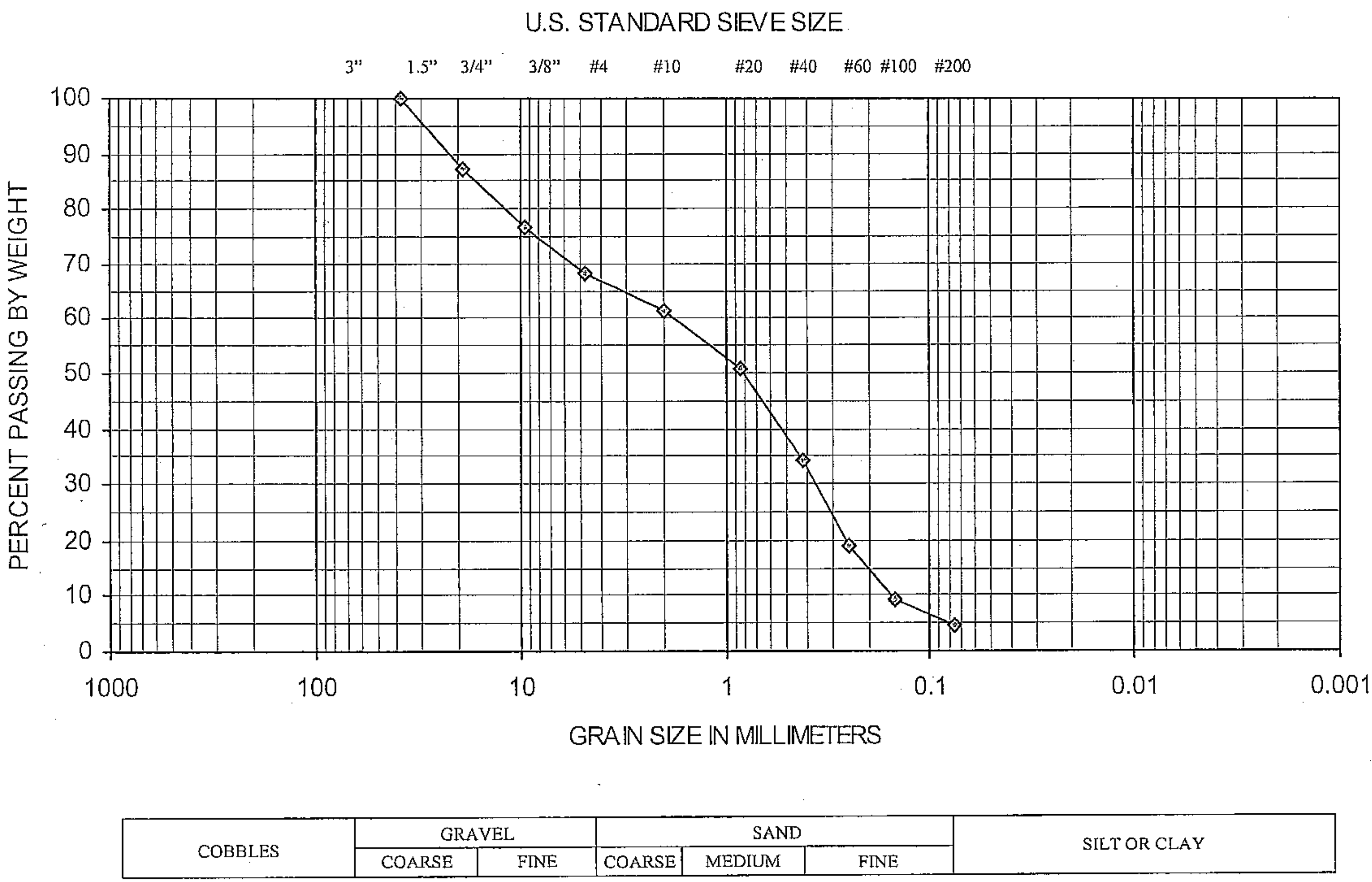
SIEVE ANALYSIS RESULTS

FIGURE B-1

0180-180-00 KGO : JVJ : jvj 8-11-05 (Sieve.ppt)



SYMBOL	EXPLORATION NUMBER	DEPTH (ft)	SOIL CLASSIFICATION
◈	TH-45-05	7	Gray fine to medium sand with gravel and trace silt (SP)

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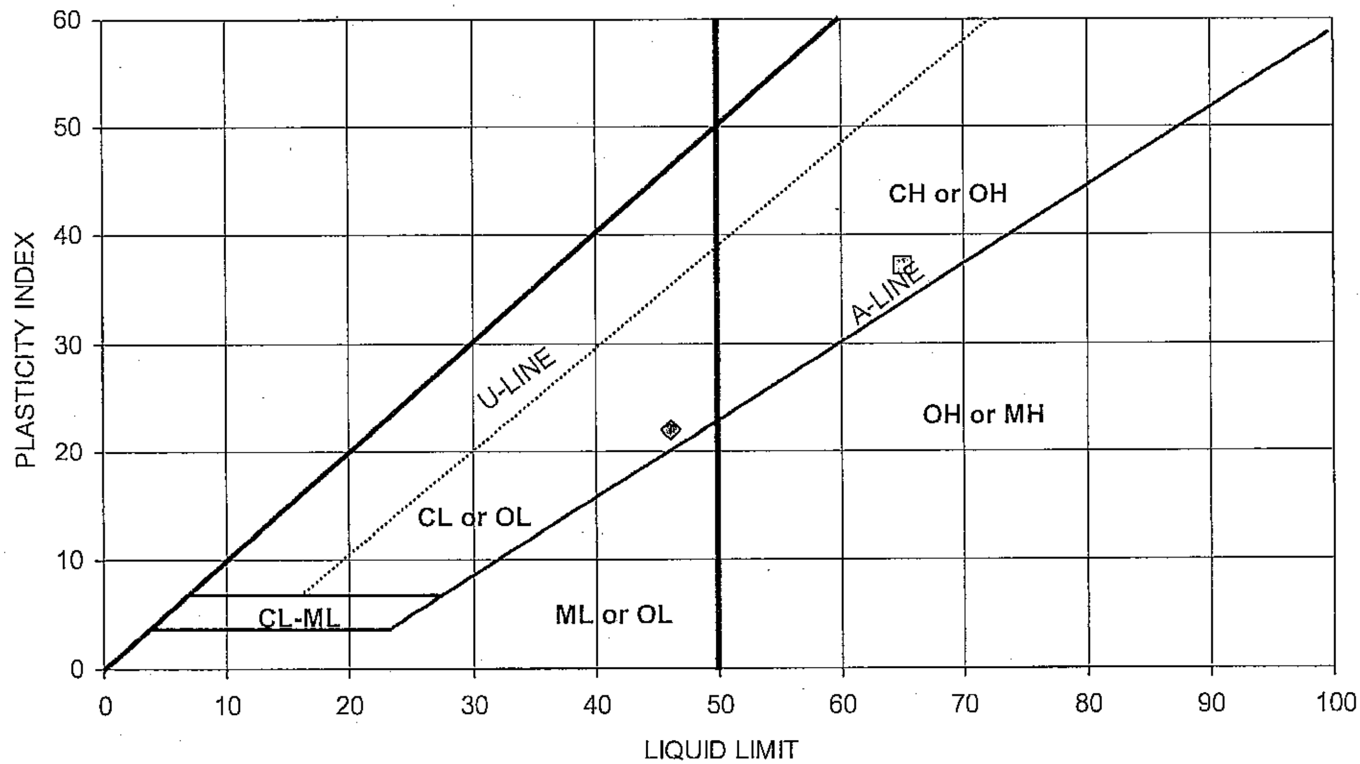
SIEVE ANALYSIS RESULTS

FIGURE B-2

0180-180-00 KGO : HMS : jvj 6-28-05 (Atterbergs.ppt)

PLASTICITY CHART



SYMBOL	EXPLORATION NUMBER	SAMPLE DEPTH	MOISTURE CONTENT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	SOIL DESCRIPTION
◈	TH-40-05	20'	29	46	22	Gray clay (CL)
⊡	TH-42-05	16'	30	65	37	Gray fat clay (CH)

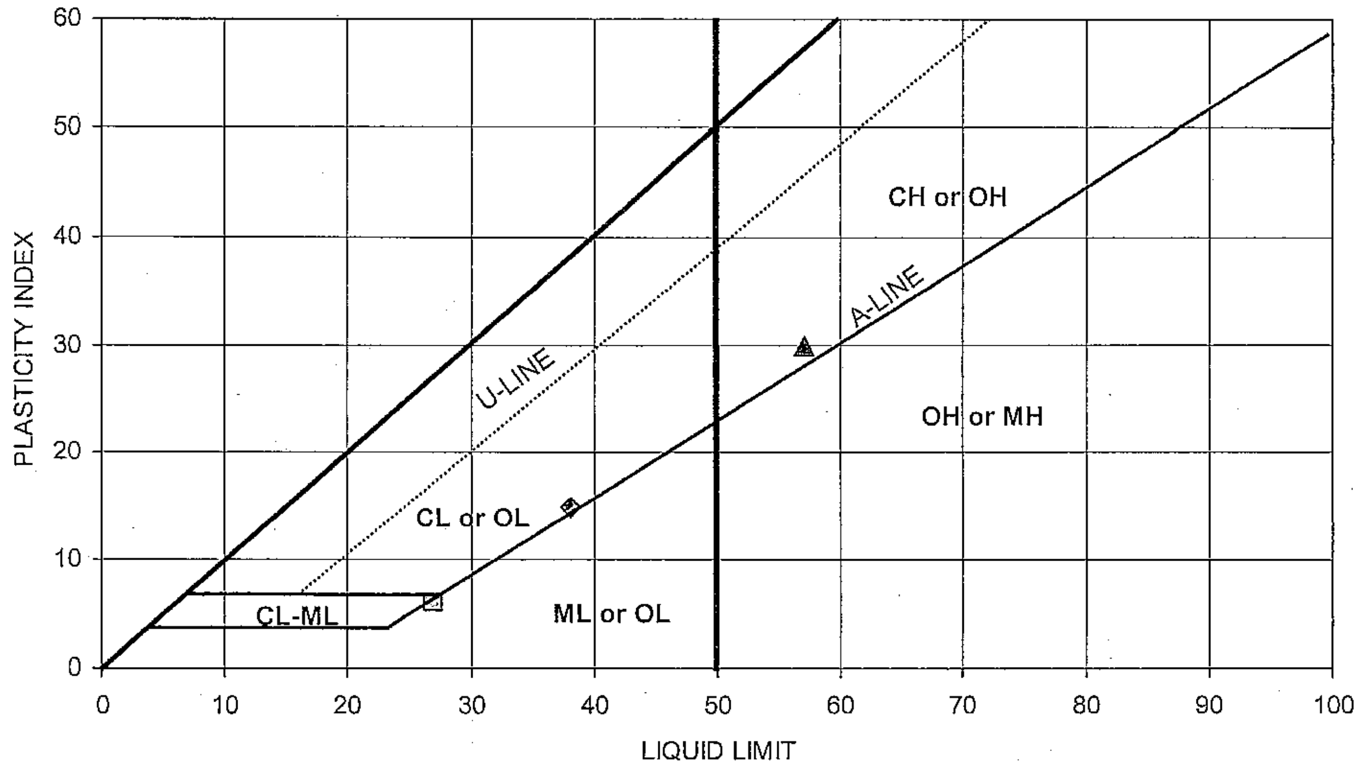
GEOENGINEERS

ATTERBERG LIMITS TEST RESULTS

FIGURE B-3

0180-180-00 KGO : CTS : cts 8-18-05 (Atterbergs.ppt)

PLASTICITY CHART

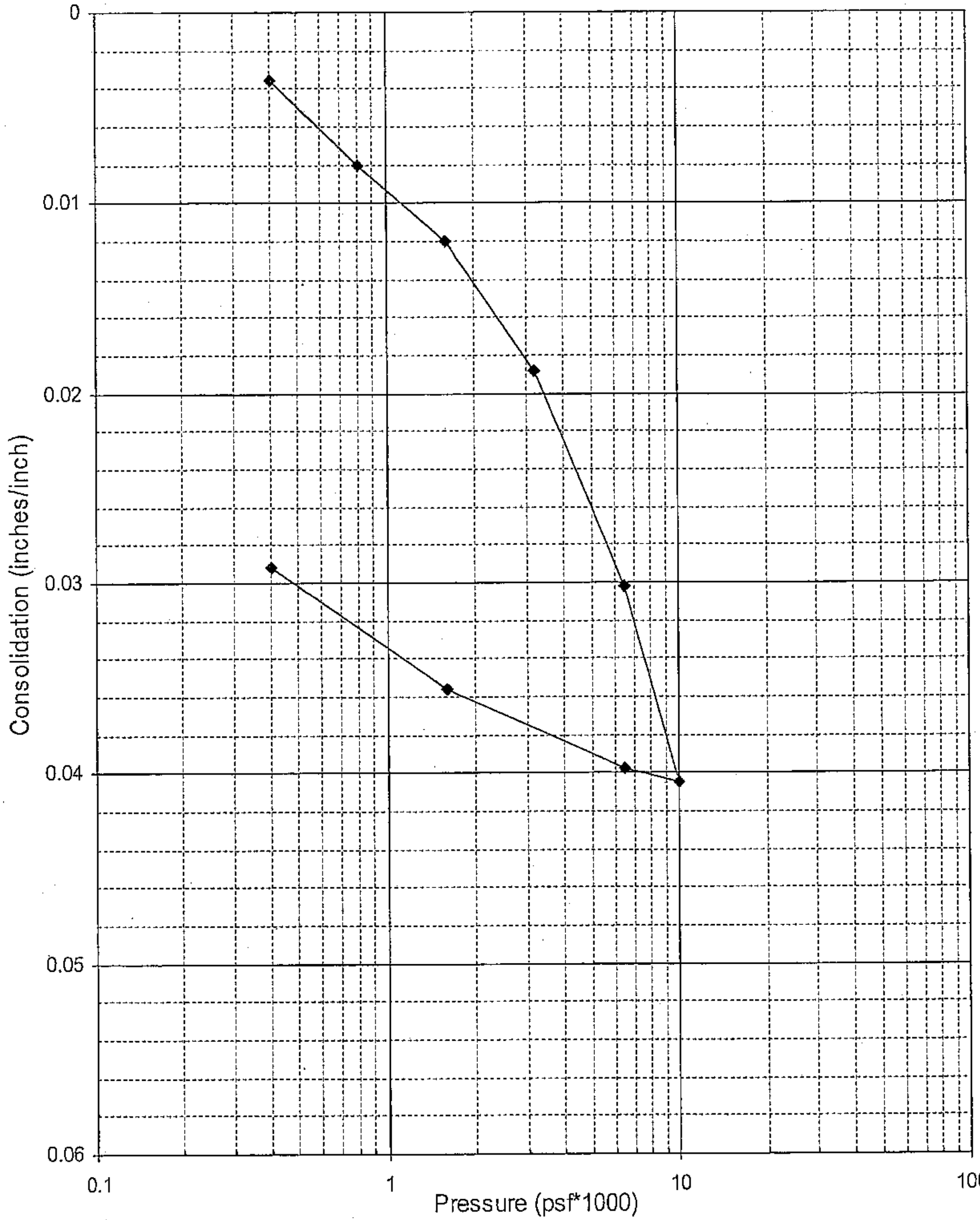


SYMBOL	EXPLORATION NUMBER	SAMPLE DEPTH	MOISTURE CONTENT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	SOIL DESCRIPTION
◈	TH-46-05	11'	29	38	15	Gray clay (CL)
⊡	TH-46-05	21'	34	27	6	Gray silt (ML)
▲	TH-46-05	40.5'	26	57	30	Gray fat clay (CH)

GEOENGINEERS

ATTERBERG LIMITS TEST RESULTS

FIGURE B-4



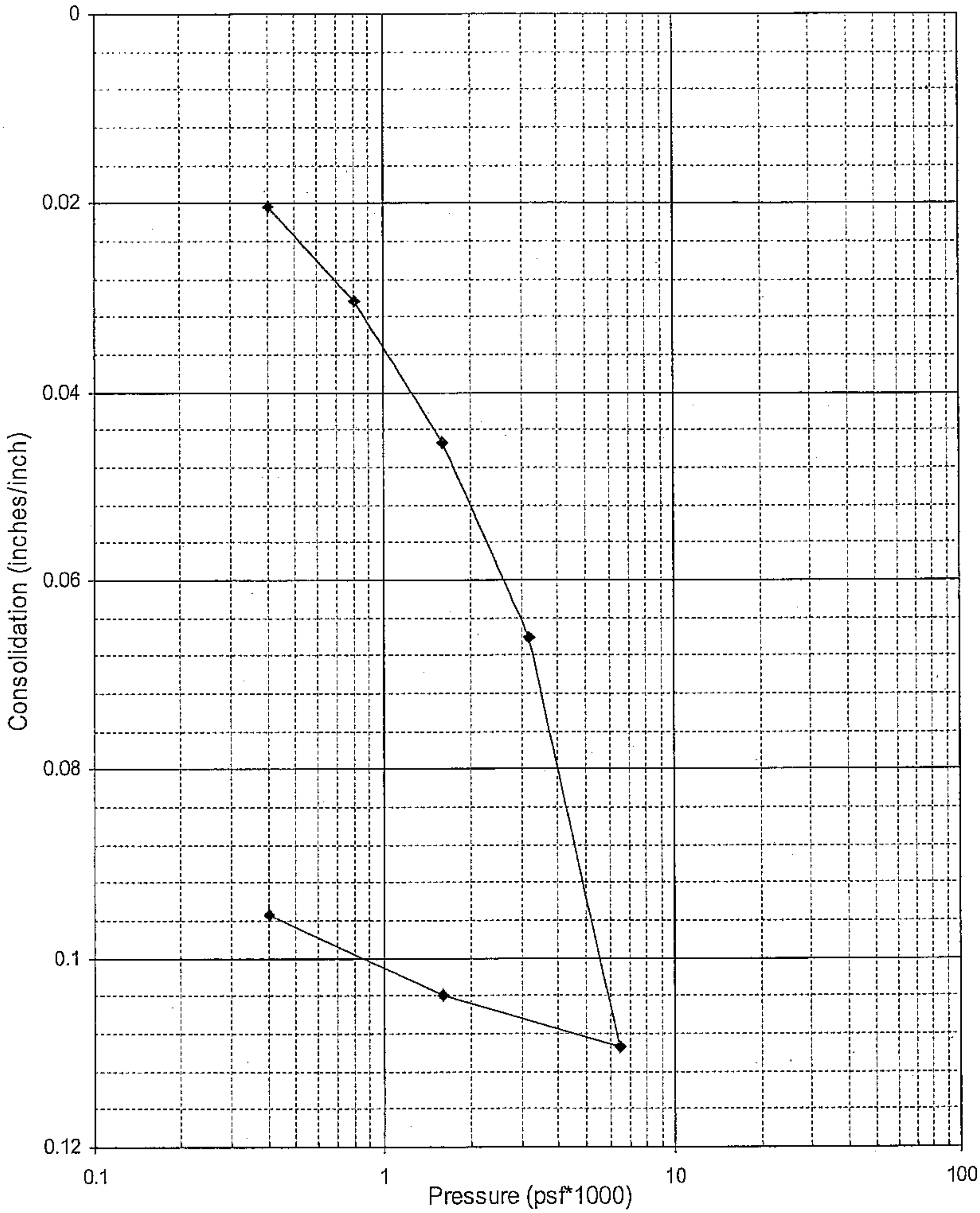
BORING NUMBER	SAMPLE DEPTH (FEET)	SOIL CLASSIFICATION	INITIAL MOISTURE CONTENT	INITIAL DRY DENSITY (LBS/FT^3)
TH-46-05	11'	Gray silt (ML)	29	97

0180-180-00 KGO : JVJ : jvj 8-19-05 (consol.ppt)

GEOENGINEERS

CONSOLIDATION TEST RESULTS

FIGURE B-5



BORING NUMBER	SAMPLE DEPTH (FEET)	SOIL CLASSIFICATION	INITIAL MOISTURE CONTENT	INITIAL DRY DENSITY (LBS/FT³)
TH-46-05	21'	Gray silt (ML)	42	81

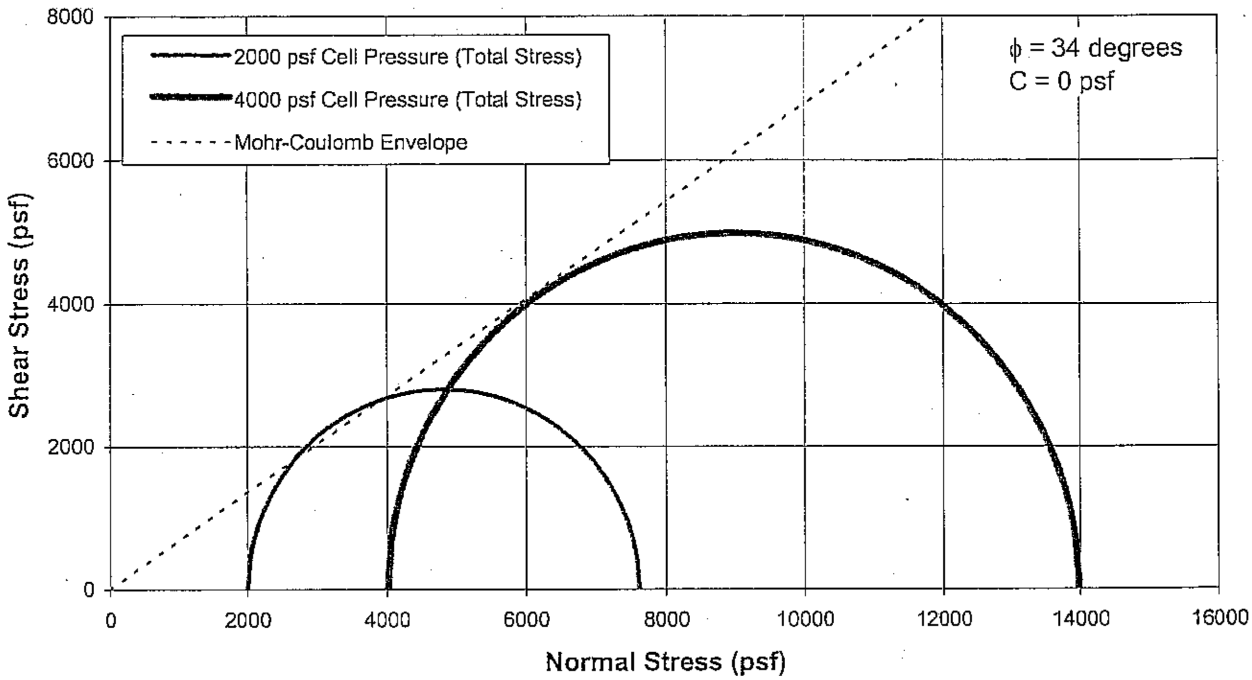
0180-180-00 KGO : JVJ : jvj 8-19-05 (consol.ppt)

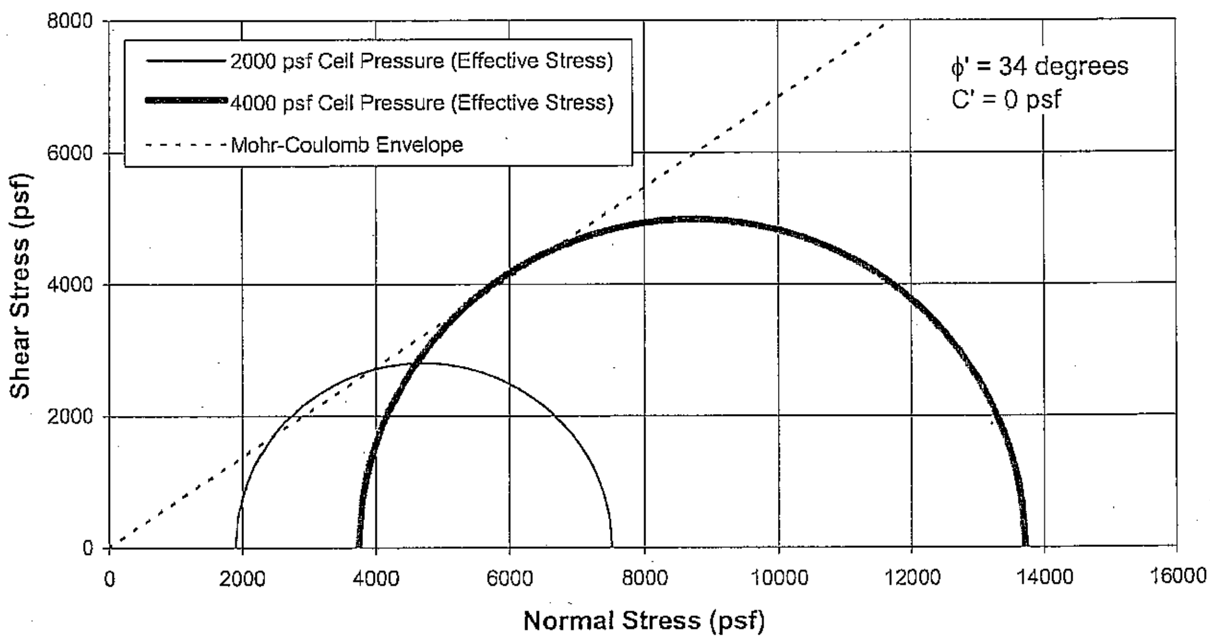
GEOENGINEERS

CONSOLIDATION TEST RESULTS

FIGURE B-6

SR 305 SCL to Bond Road
Staged Triaxial Consolidated Undrained (CU) Compression Test





Sample: TH-46-05 (S-5)
Strain Rate (%/min): 0.1
Sample Back Pressure Saturated at: 5760 (psf)
Initial B value = 0.98

Sample Description: Gray clay (CL)
Failure Defined at Peak Stress
Initial Sample Height (in): 6.76

0180-180-00 BJM:KGO:DJC P:\0\0180180\00\lab\TXCU1.xls

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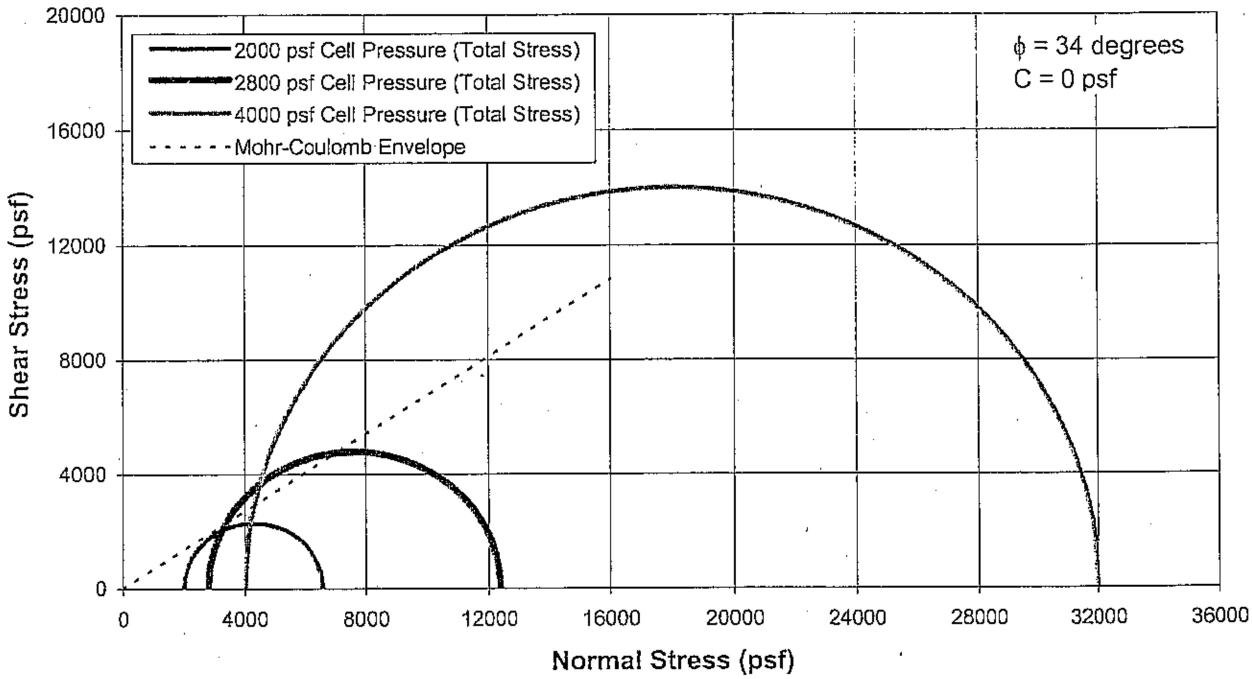
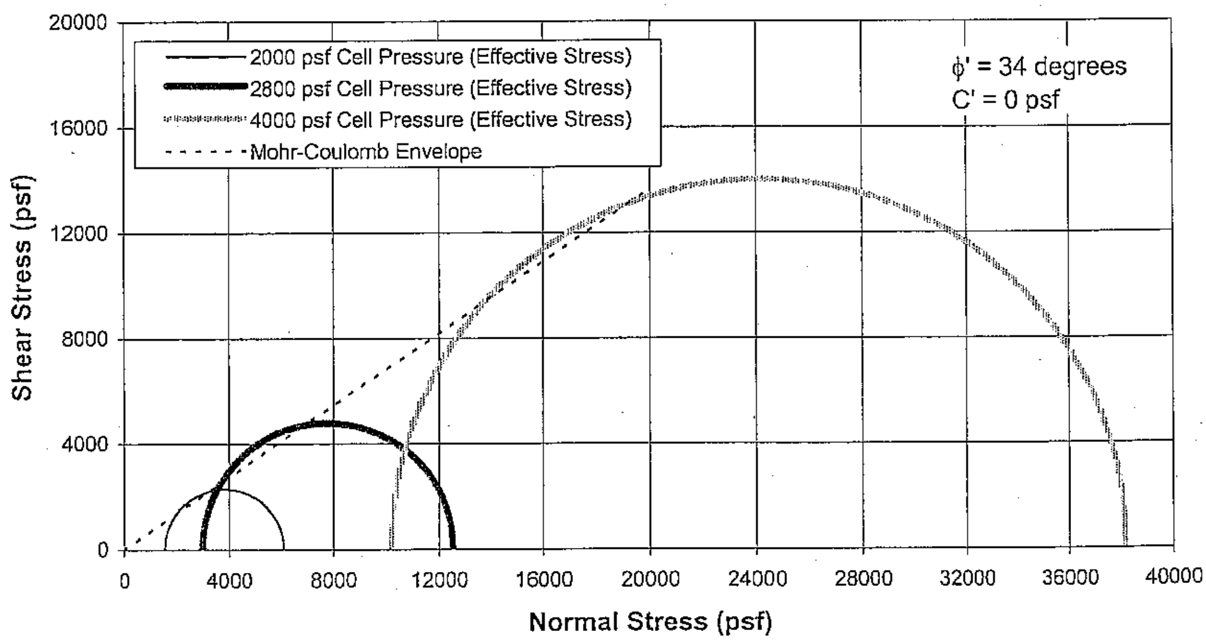
Earth Science + Technology

Consolidated Undrained Triaxial Test Results

FIGURE B-7

SR 305 SCL to Bond Road
Staged Triaxial Consolidated Undrained (CU) Compression Test





Sample: TH-46-05 (S-9)
Strain Rate (%/min): 0.1
Sample Back Pressure Saturated at: 7200 (psf)
Initial B value = 0.98

Sample Description: Gray silt (ML)
Failure Defined at Peak Stress
Initial Sample Height (in): 5.61

0180-180-00 BJM:KGO:DJC: P:\01\0180180\00\lab\TXCU2.xls

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Consolidated Undrained Triaxial Test Results

FIGURE B-8

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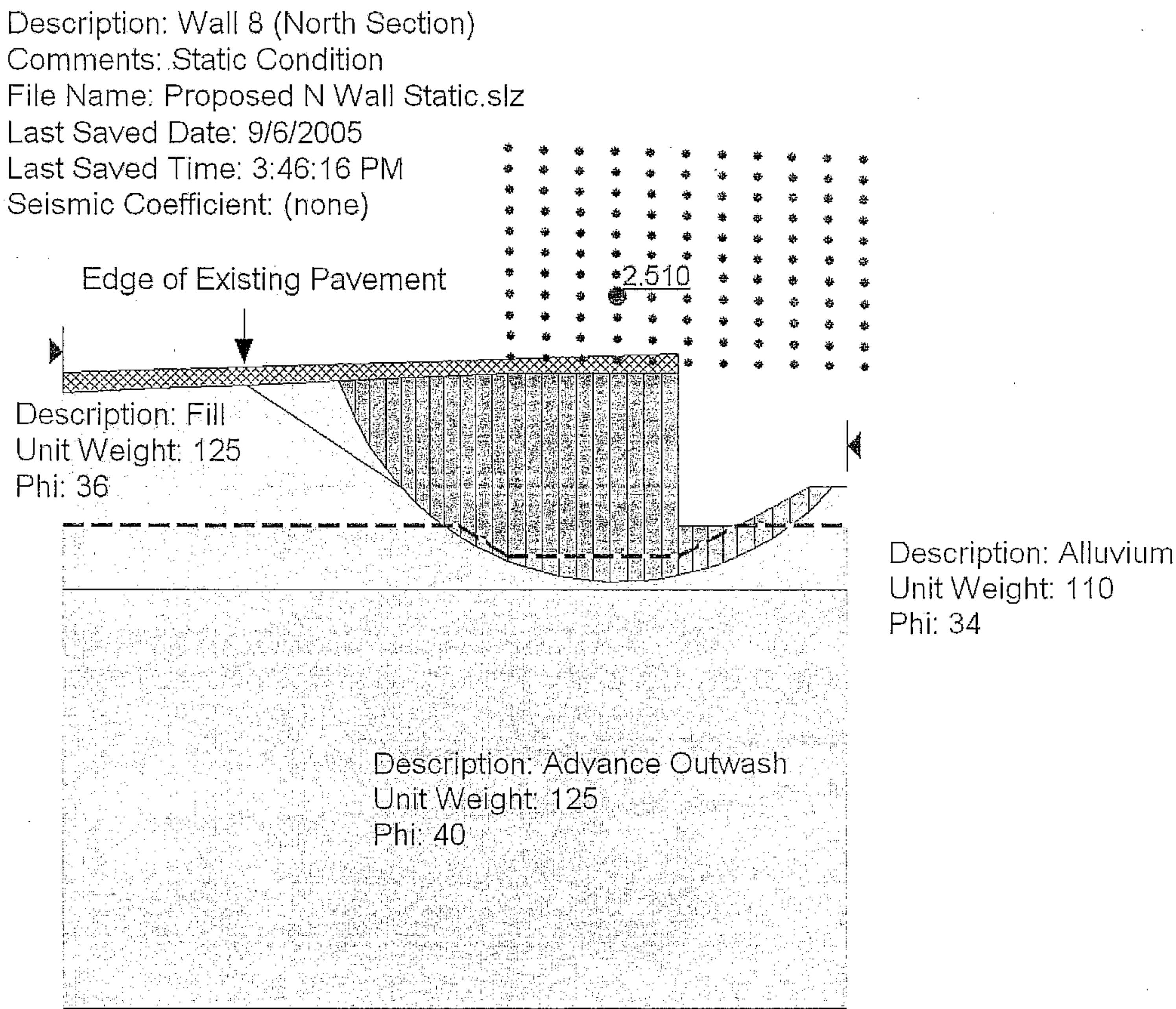
Appendix C

Wall Slope Stability Results

0180-180-00 TB2:KGO 9-6-05 P:0\0180180\00\Working\Slope Stability Analysis\Slope Stability figures.ppt



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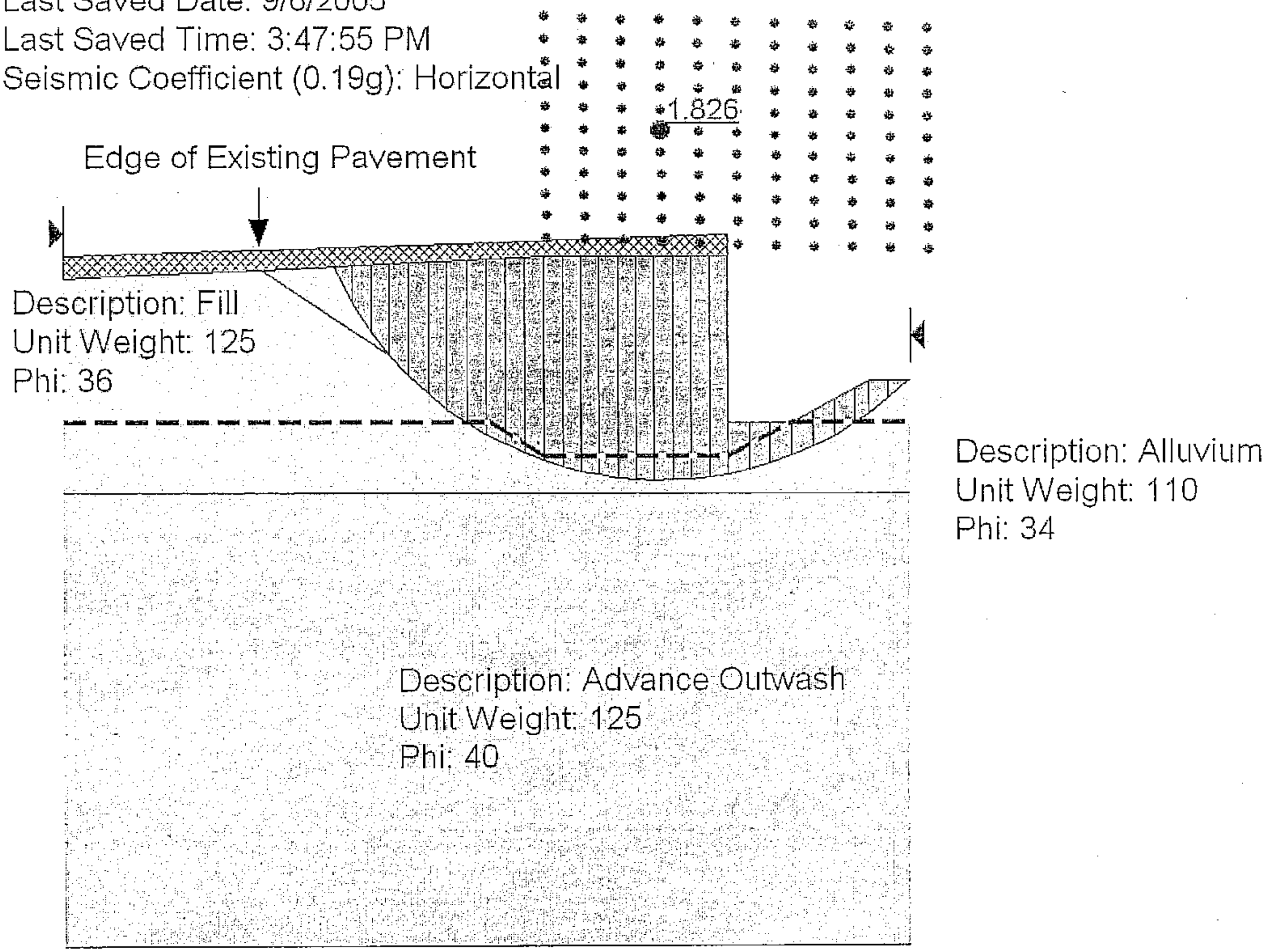
Wall 8 (North Section) Static Condition

FIGURE C-1

Description: Wall 8 (North Section)
Comments: Seismic (Pseudo Static) Condition
File Name: Proposed N Wall Seismic.slz
Last Saved Date: 9/6/2005
Last Saved Time: 3:47:55 PM
Seismic Coefficient (0.19g): Horizontal



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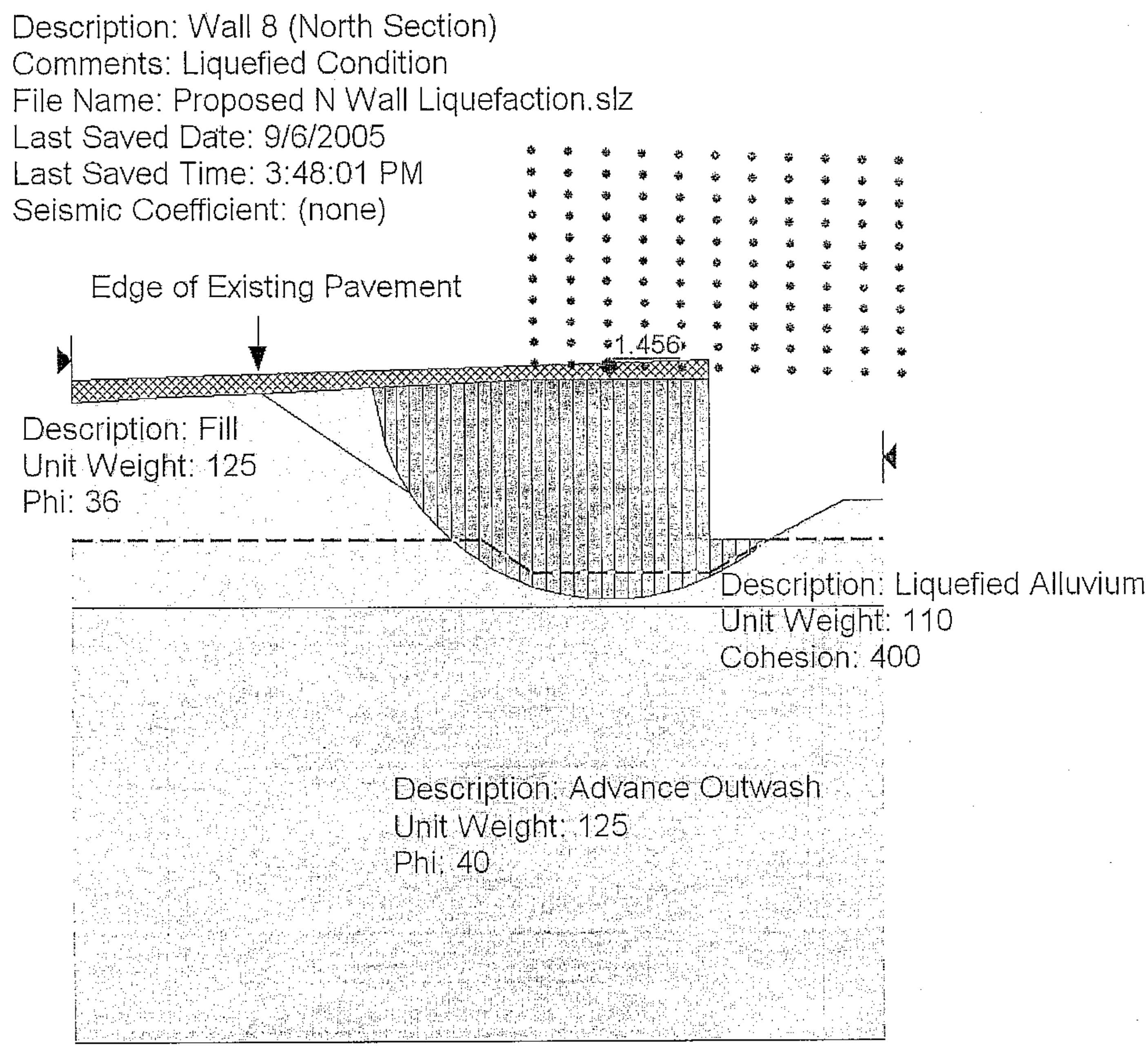
Wall 8 (North Section) Pseudo Static Condition

FIGURE C-2



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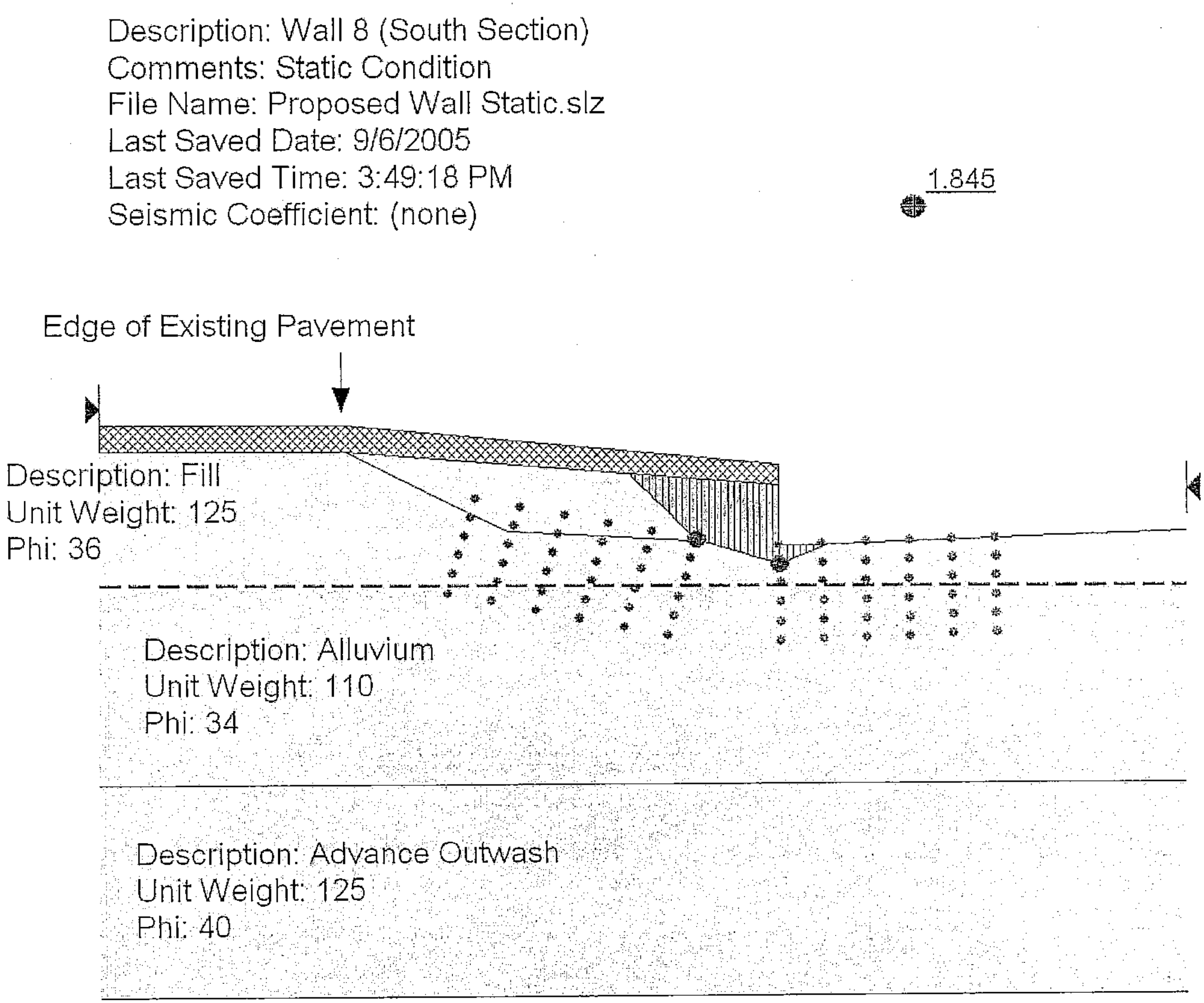
Wall 8 (North Section) Liquefied Condition

FIGURE C-3



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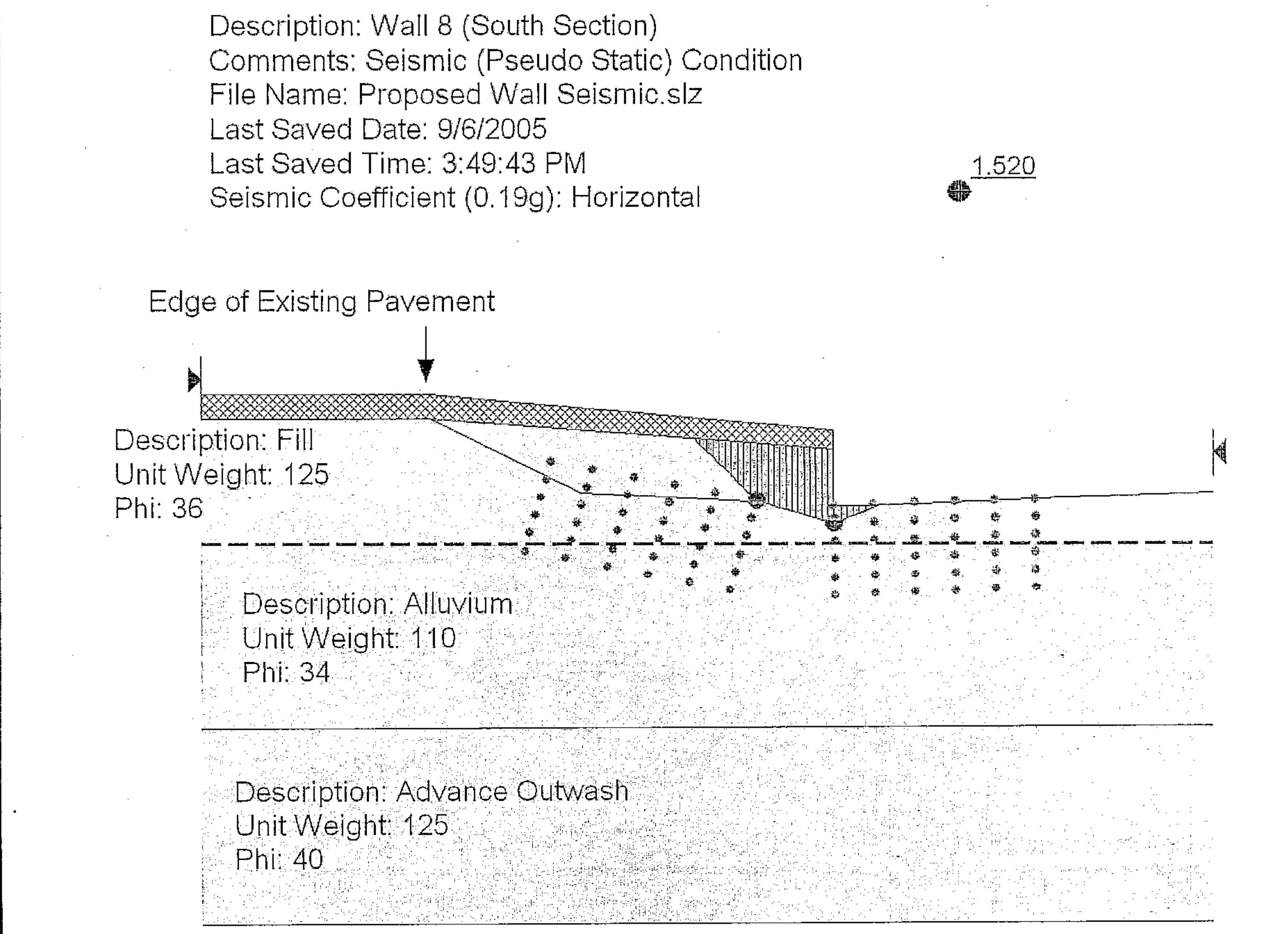
Wall 8 (South Section) Static Condition

FIGURE C-4

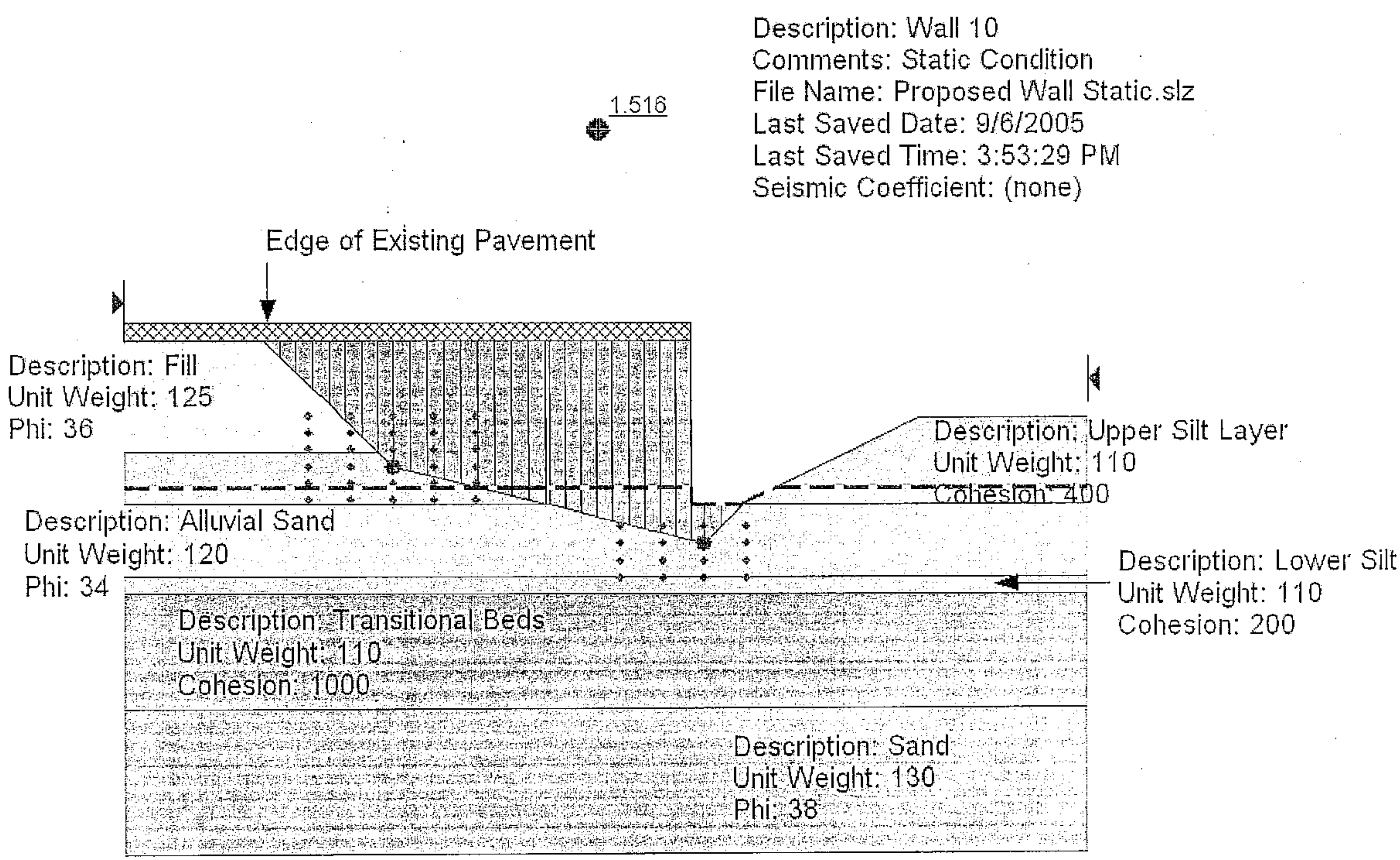
0180-180-00 TB2:KGO 9-6-05 P:0\0180180\00\Working\Slope Stability Analysis\Slope Stability figures.ppt



GEOENGINEERS

Wall 8 (South Section) Pseudo Static Condition

FIGURE C-5



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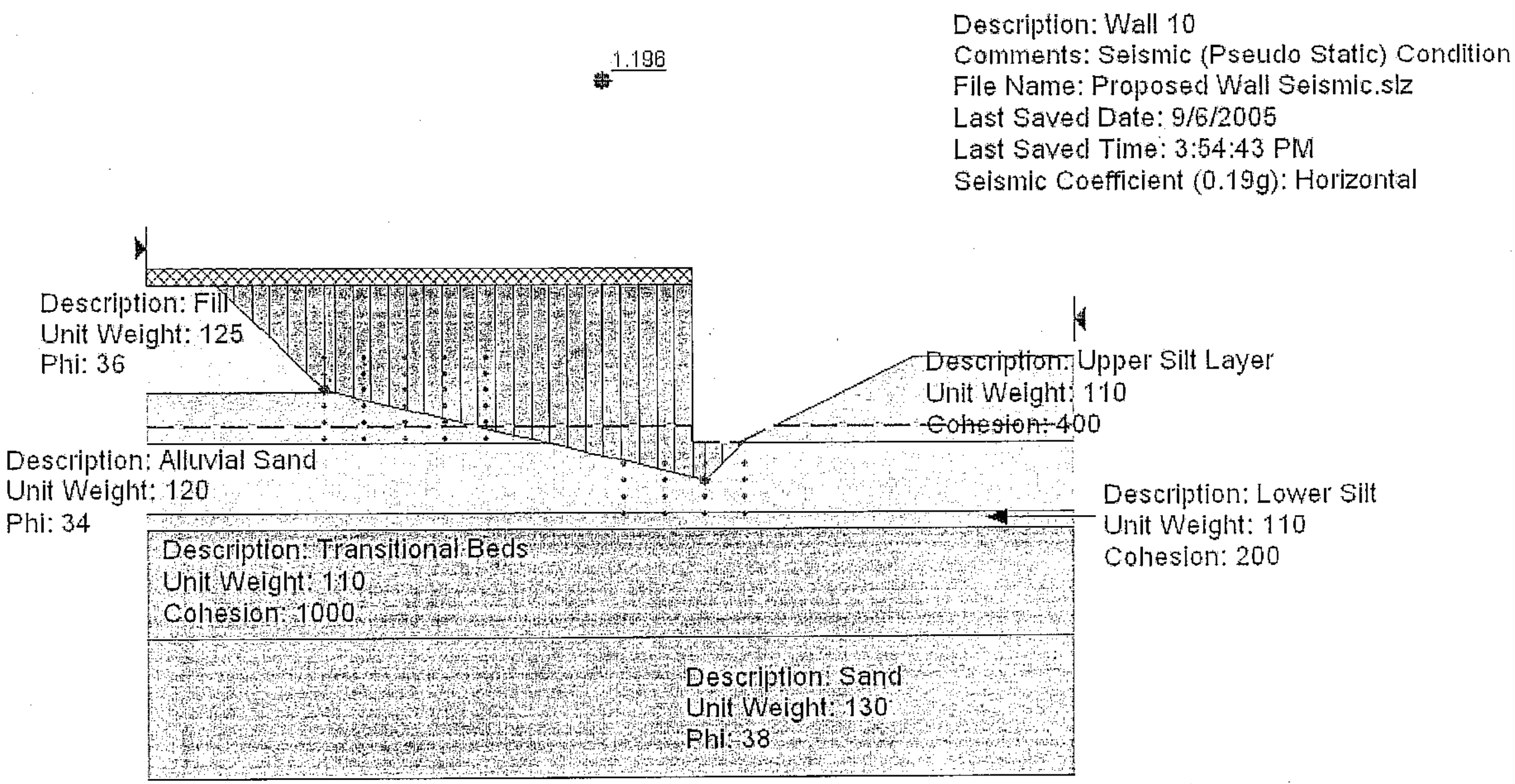
Wall 10 Static Condition

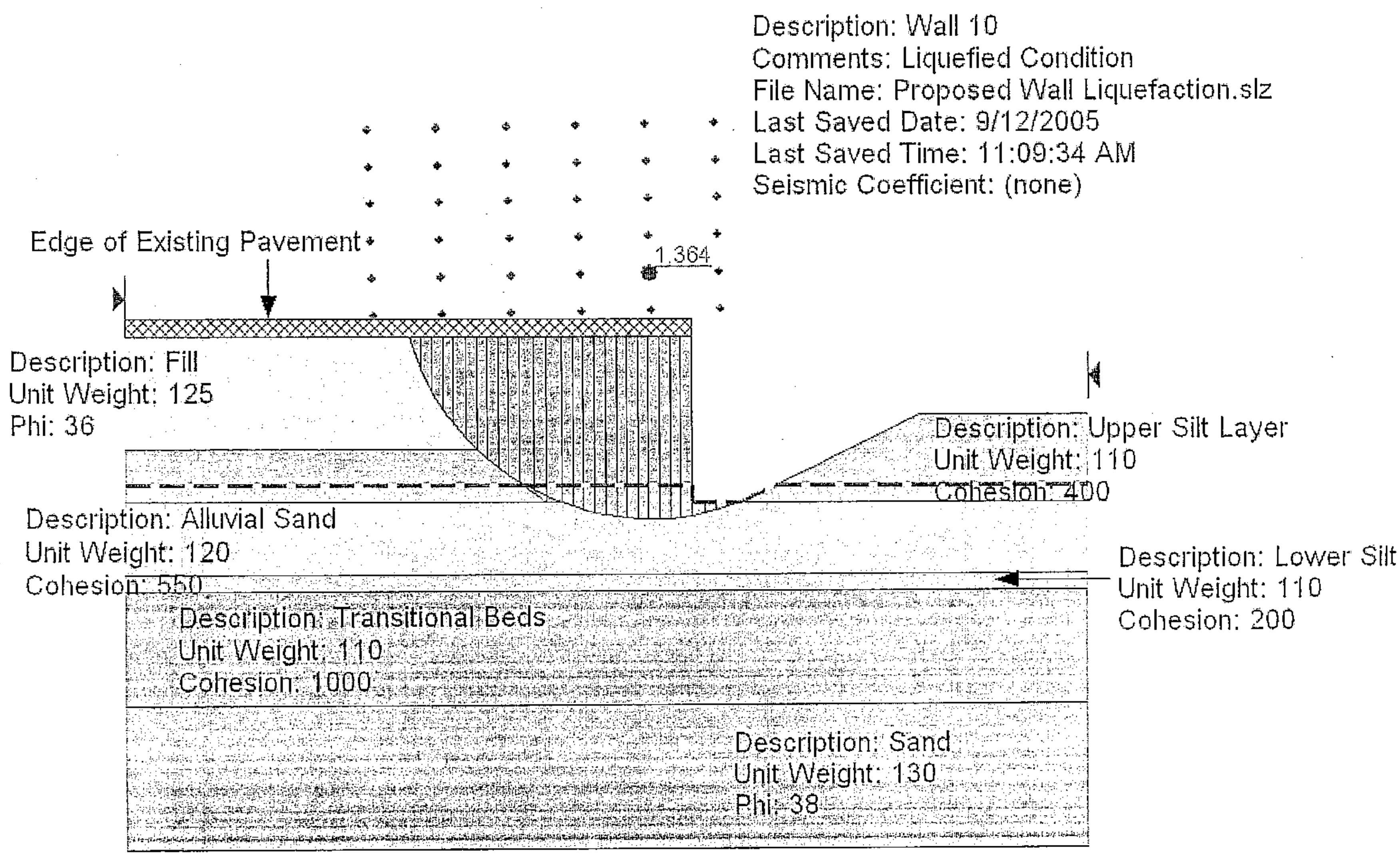
FIGURE C-6



GEOENGINEERS

Wall 10 Pseudo Static Condition

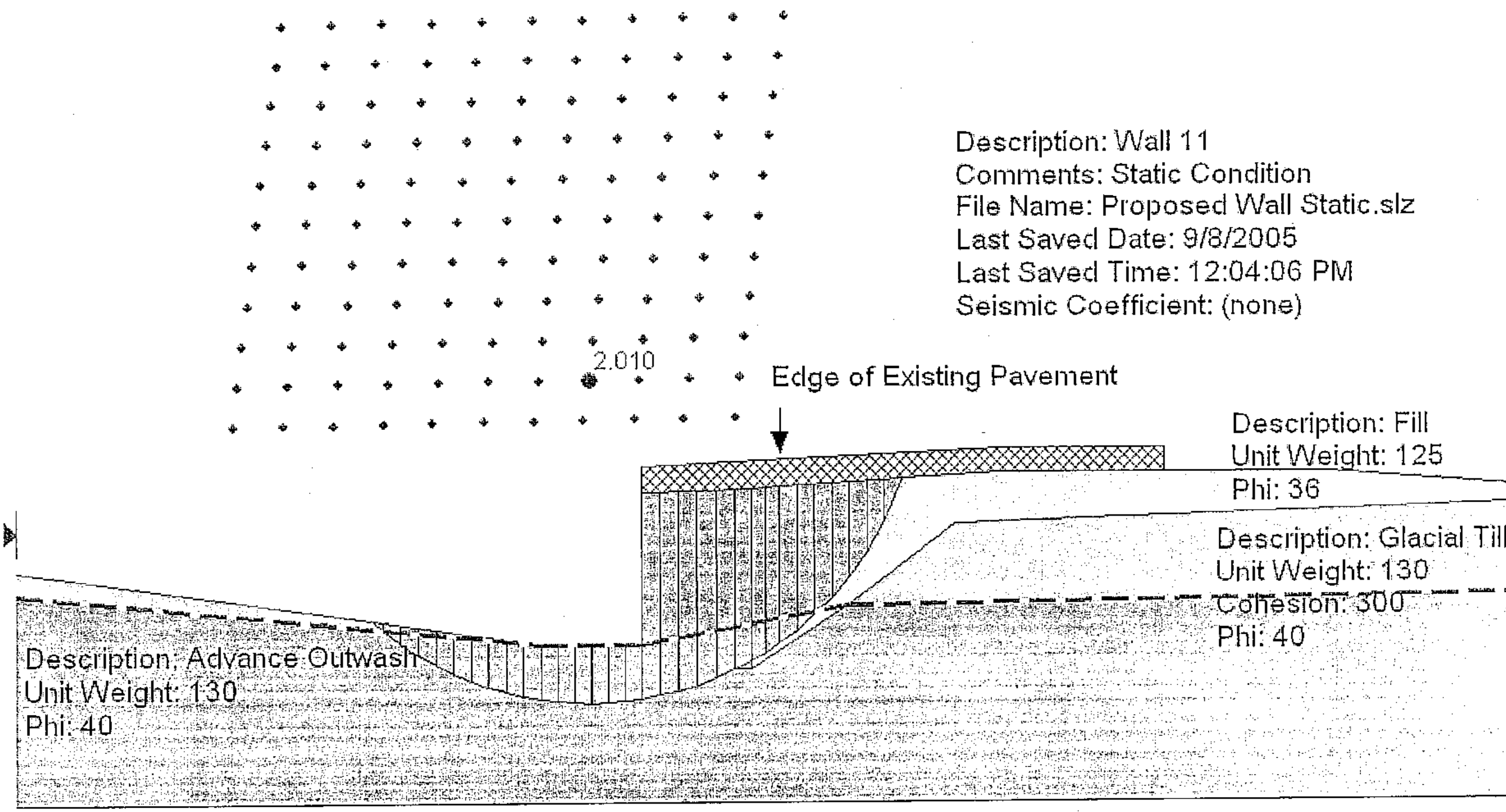
FIGURE C-7



GEOENGINEERS

Wall 10 Liquefied Condition

FIGURE C-8



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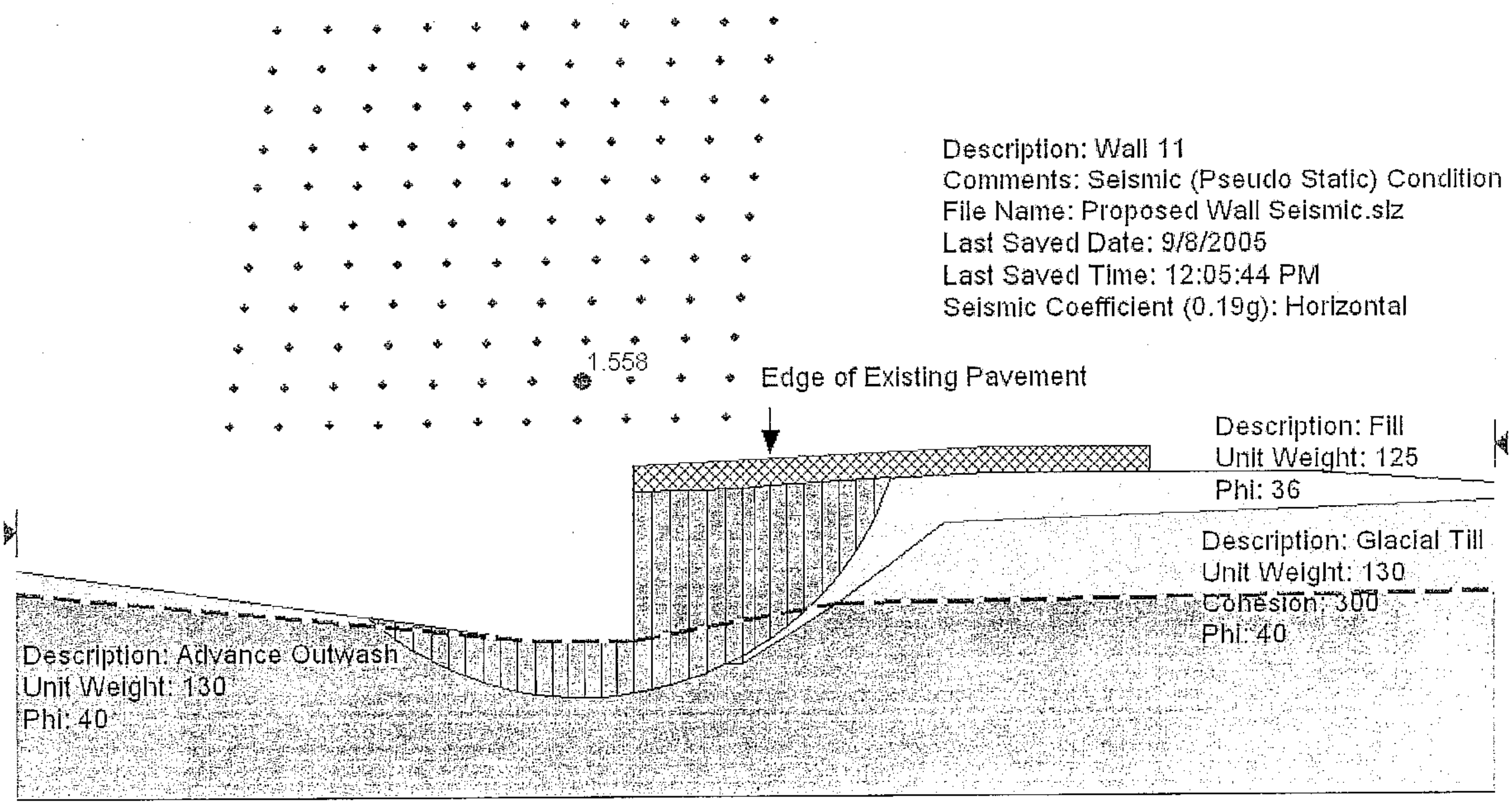
Wall 11 Static Condition

FIGURE C-9

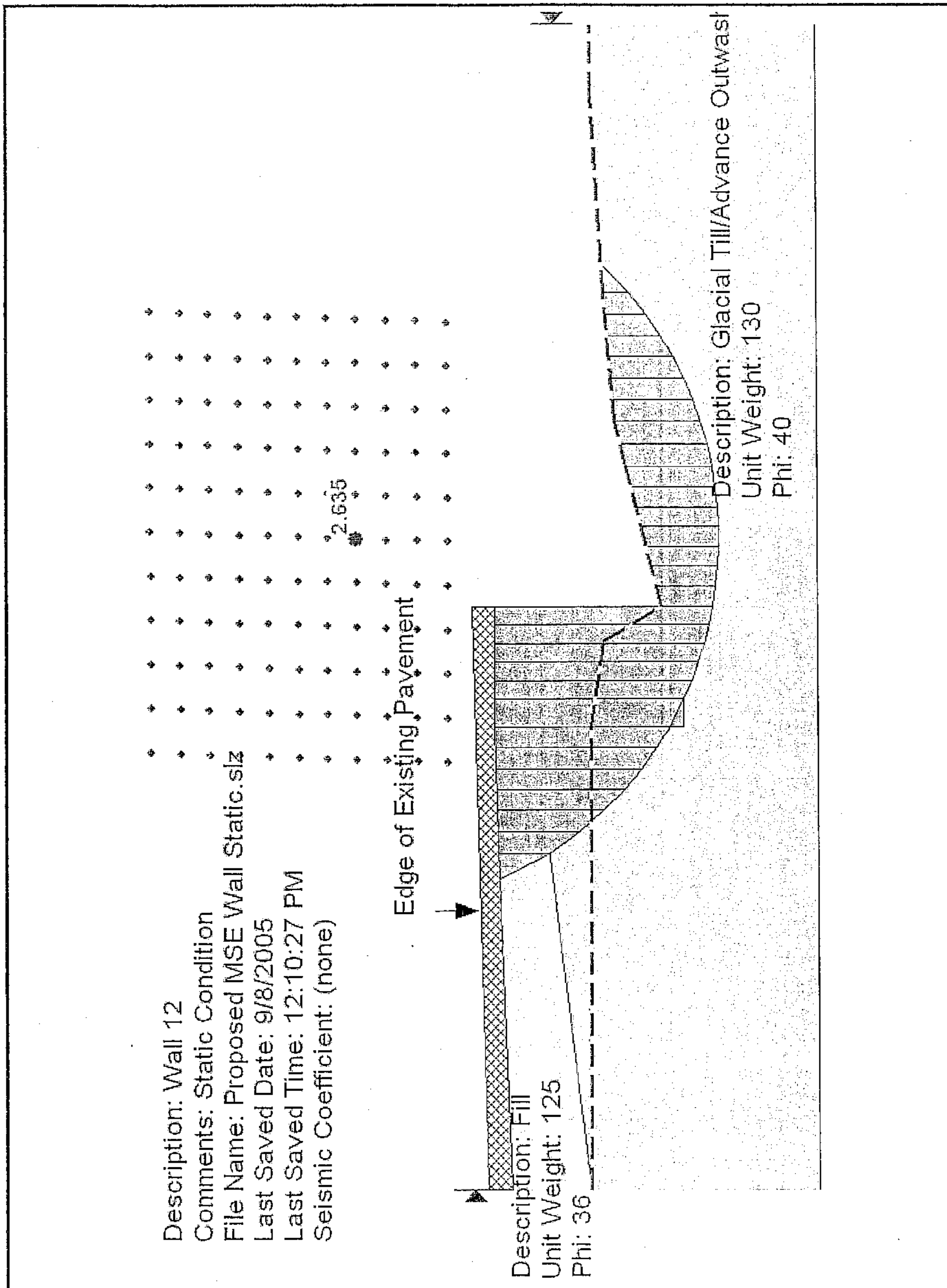


GEOENGINEERS

Wall 11 Pseudo Static Condition

FIGURE C-10

0180-180-00 TB2:KGO 9-6-05 P:\0\0180180\00\Working\Slope Stability Analysis\Slope Stability figures.ppt



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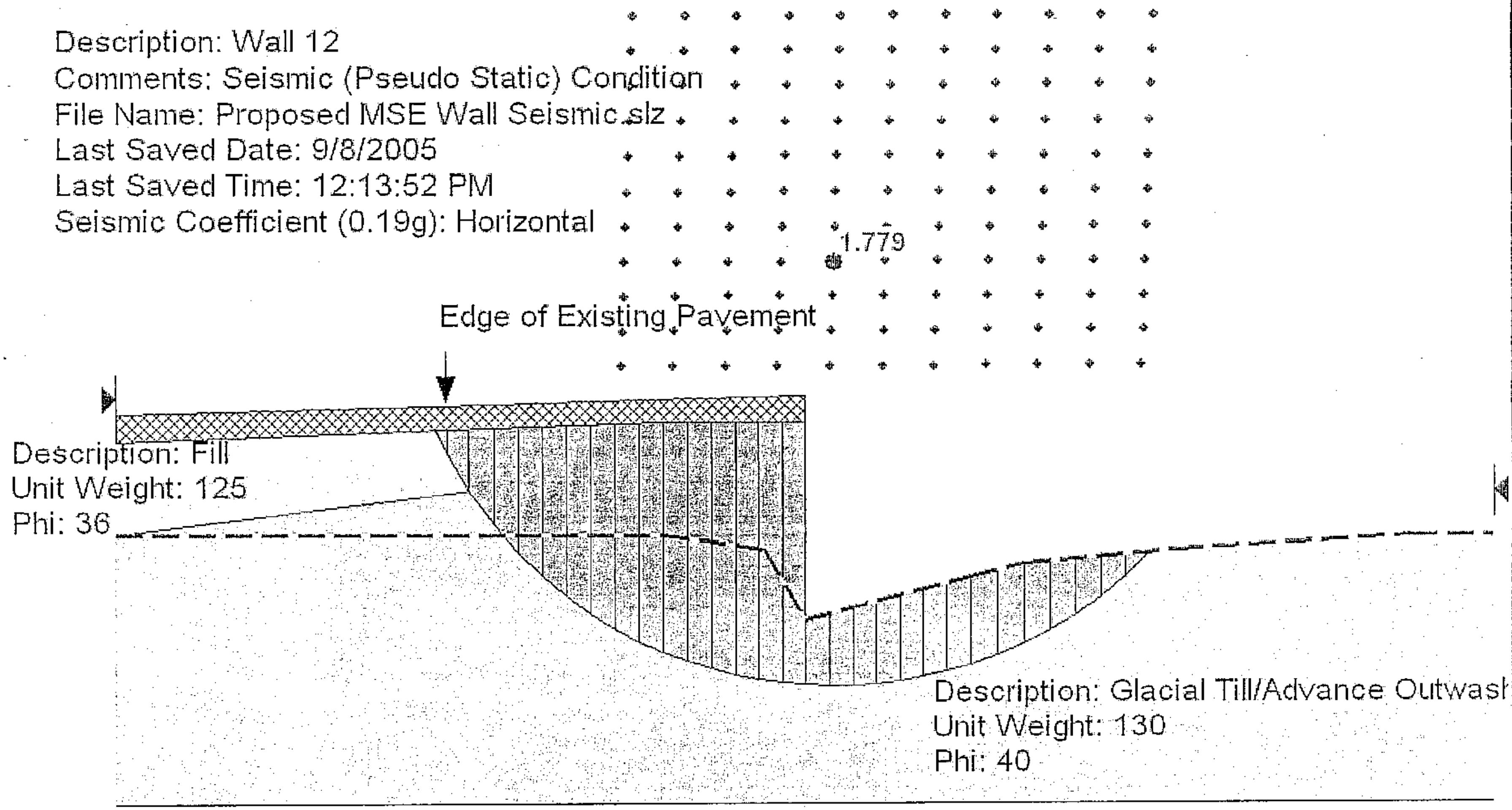
Wall 12 Static Condition

FIGURE C-11



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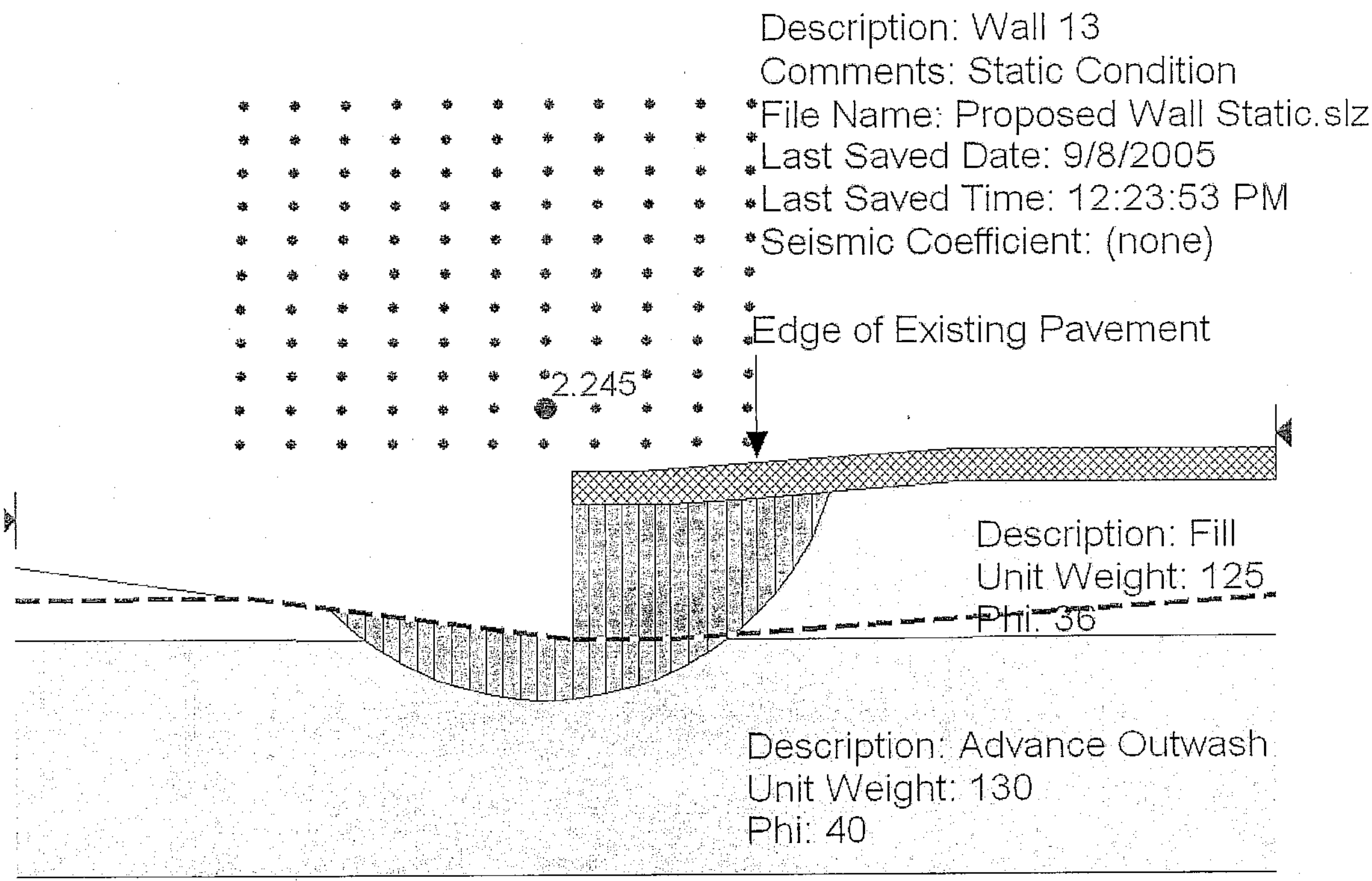
Wall 12 Pseudo Static Condition

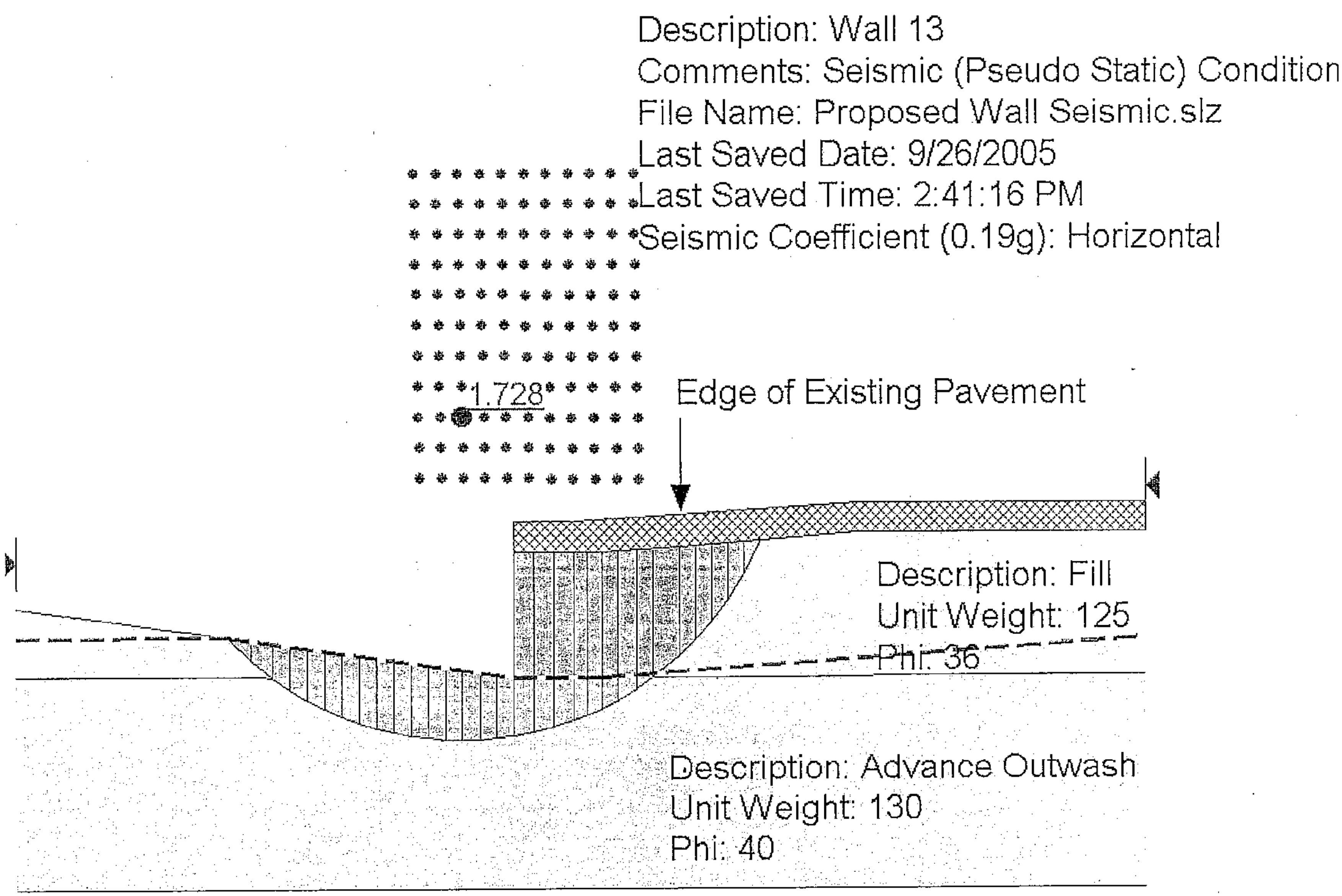
FIGURE C-12



GEOENGINEERS

Wall 13 Static Condition

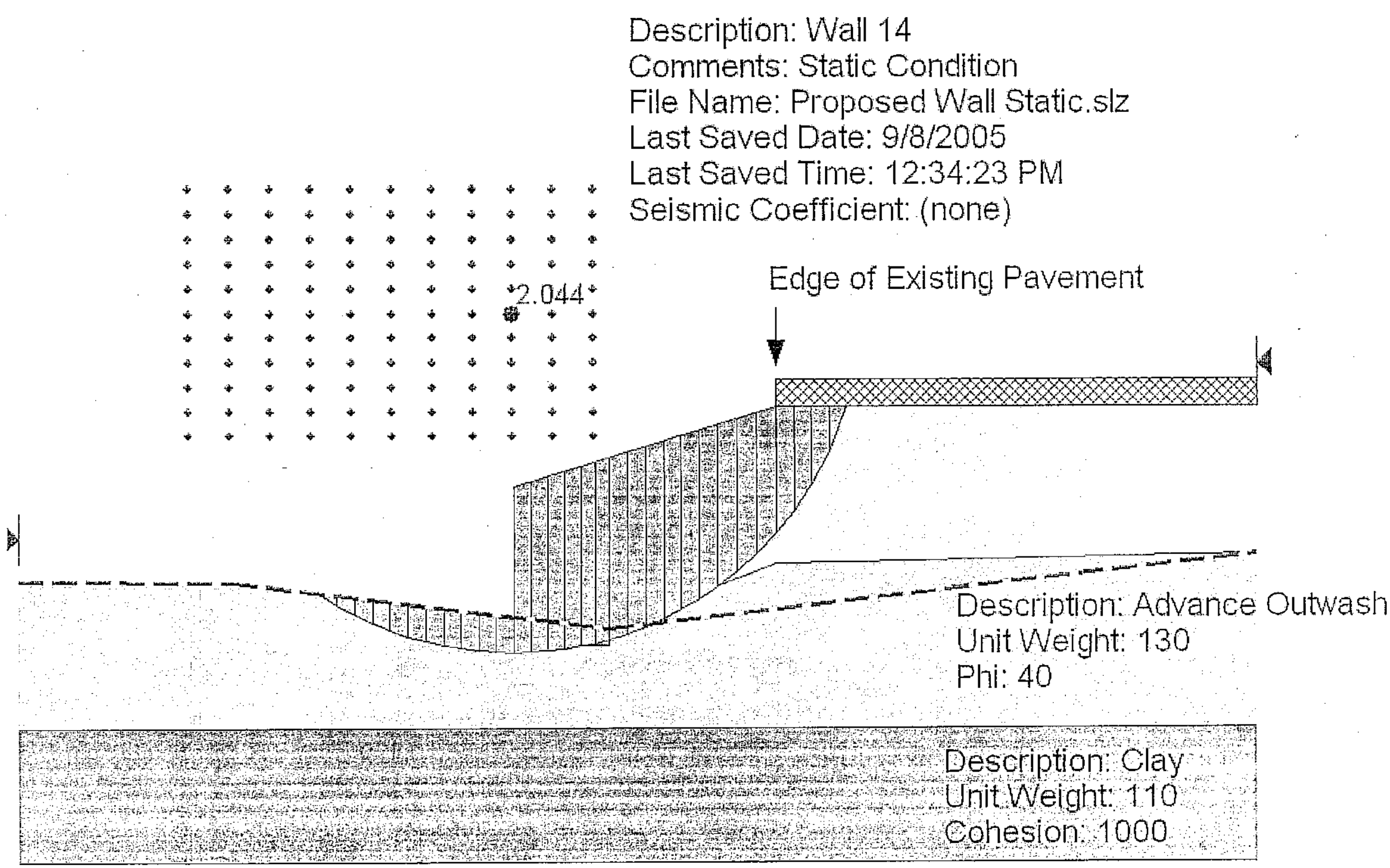
FIGURE C-13



GEOENGINEERS

Wall 13 Pseudo Static Condition

FIGURE C-14



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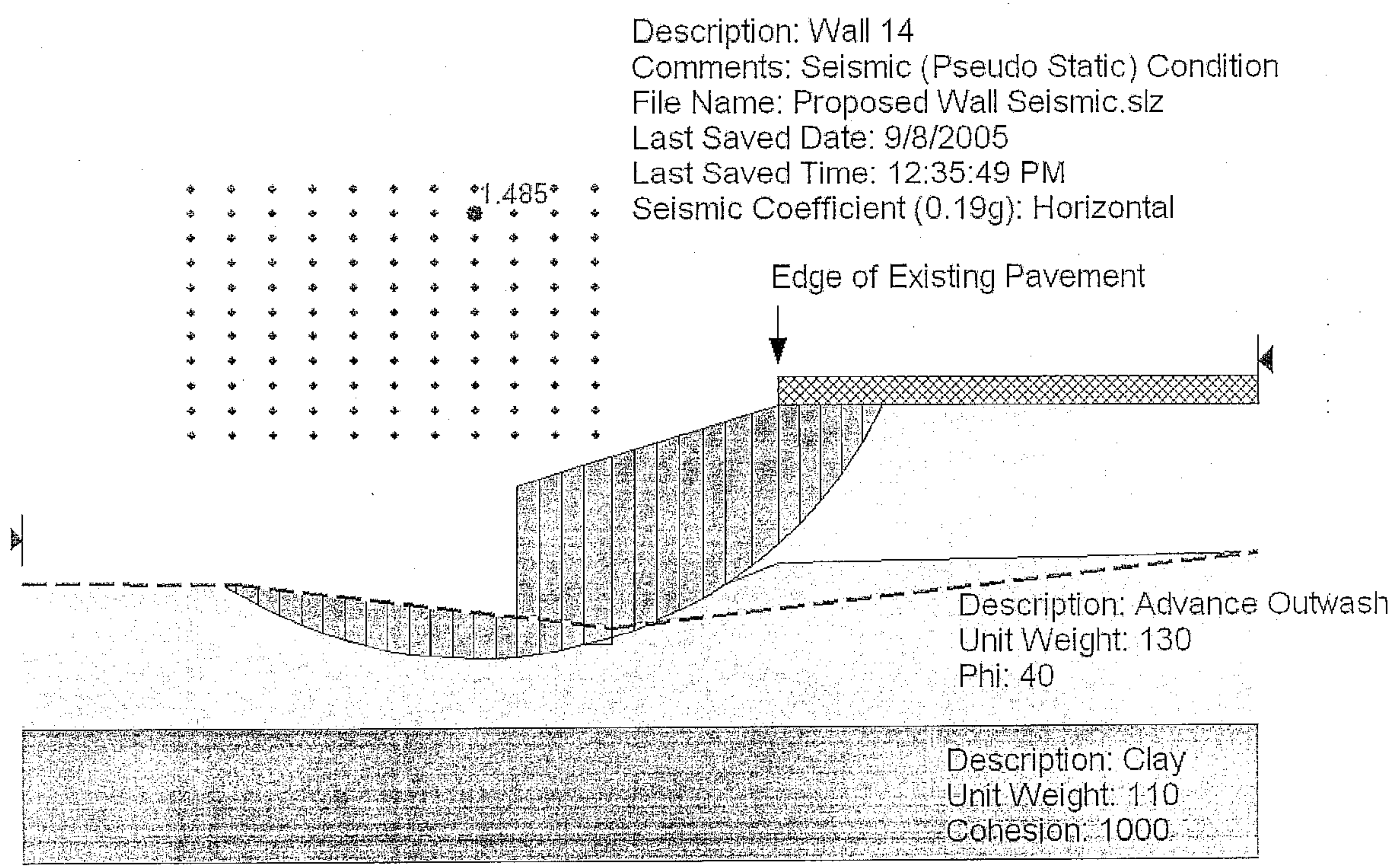
Wall 14 Static Condition

FIGURE C-15

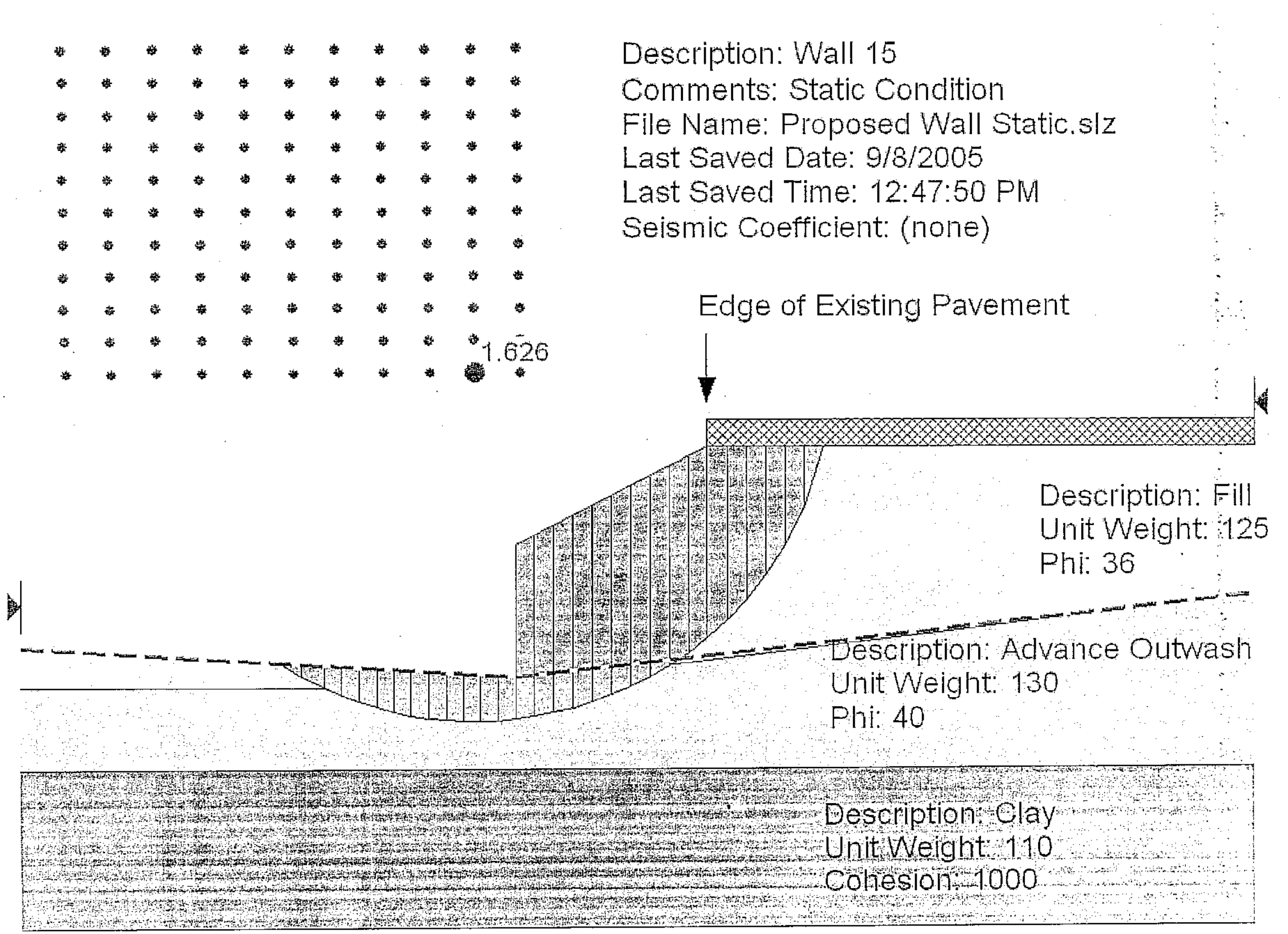


GEOENGINEERS

Wall 14 Pseudo Static Condition

FIGURE C-16

0180-180-00 TB2:KGO 9-6-05 P:0\0180180\00\Working\Slope Stability Analysis\Slope Stability figures.ppt

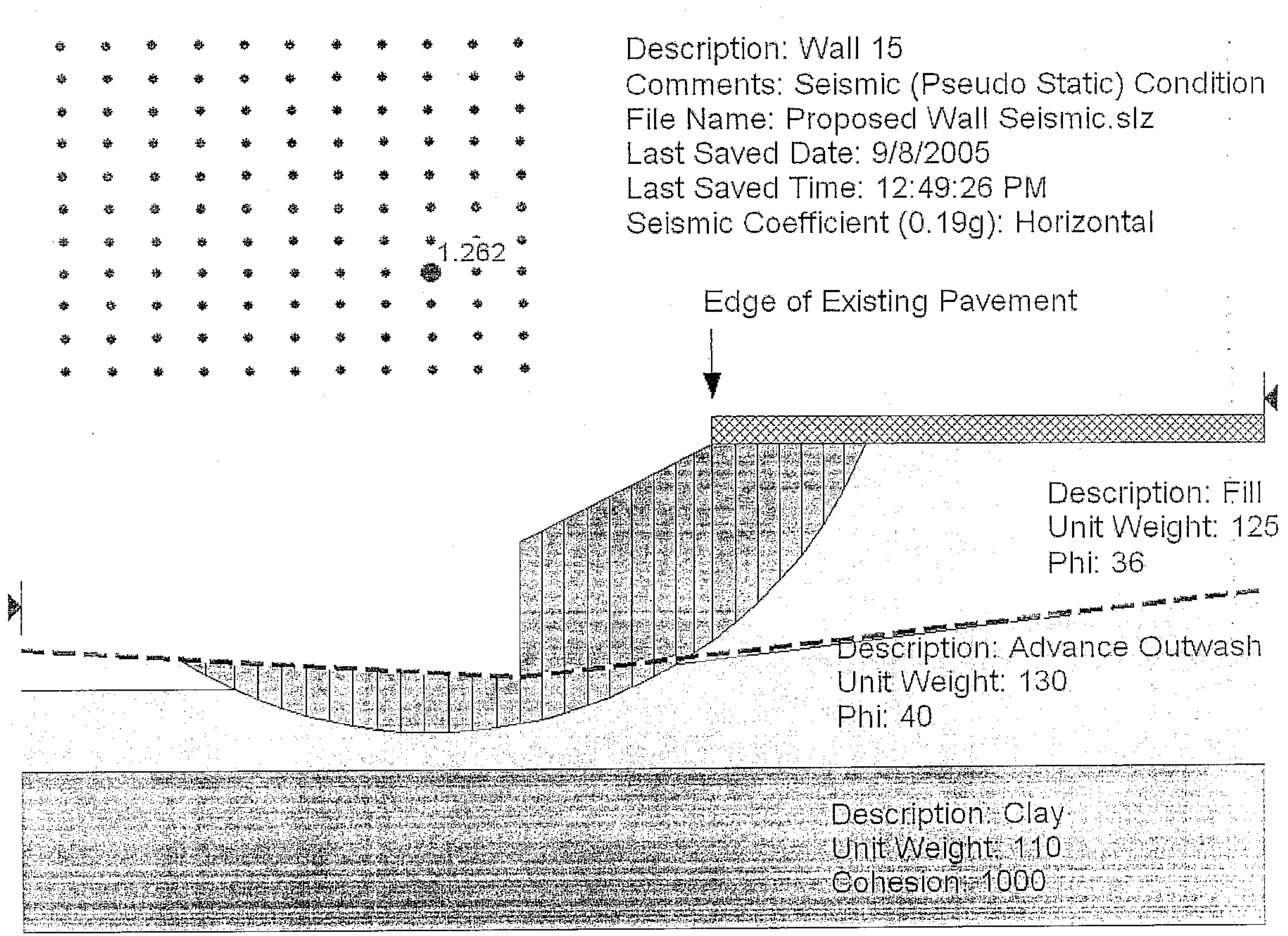


GEOENGINEERS

Wall 15 Static Condition

FIGURE C-17

0180-180-00 TB2:KGO 9-6-05 P:0\0180180\00\Working\Slope Stability Analysis\Slope Stability figures.ppt

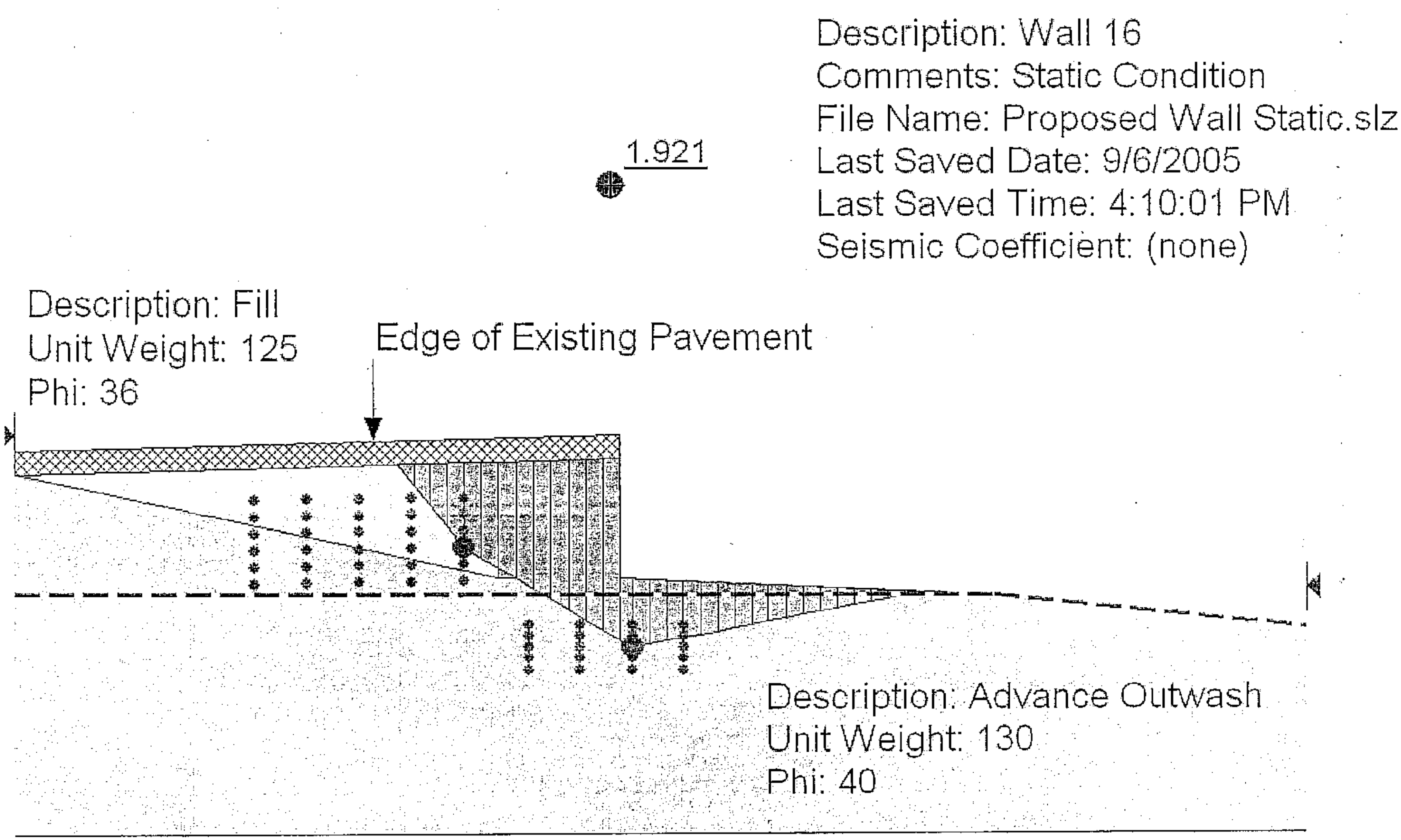


GEOENGINEERS

Wall 15 Pseudo Static Condition

FIGURE C-18

0180-180-00 TB2:KGO 9-6-05 P:0\0180180\00\Working\Slope Stability Analysis\Slope Stability figures.ppt

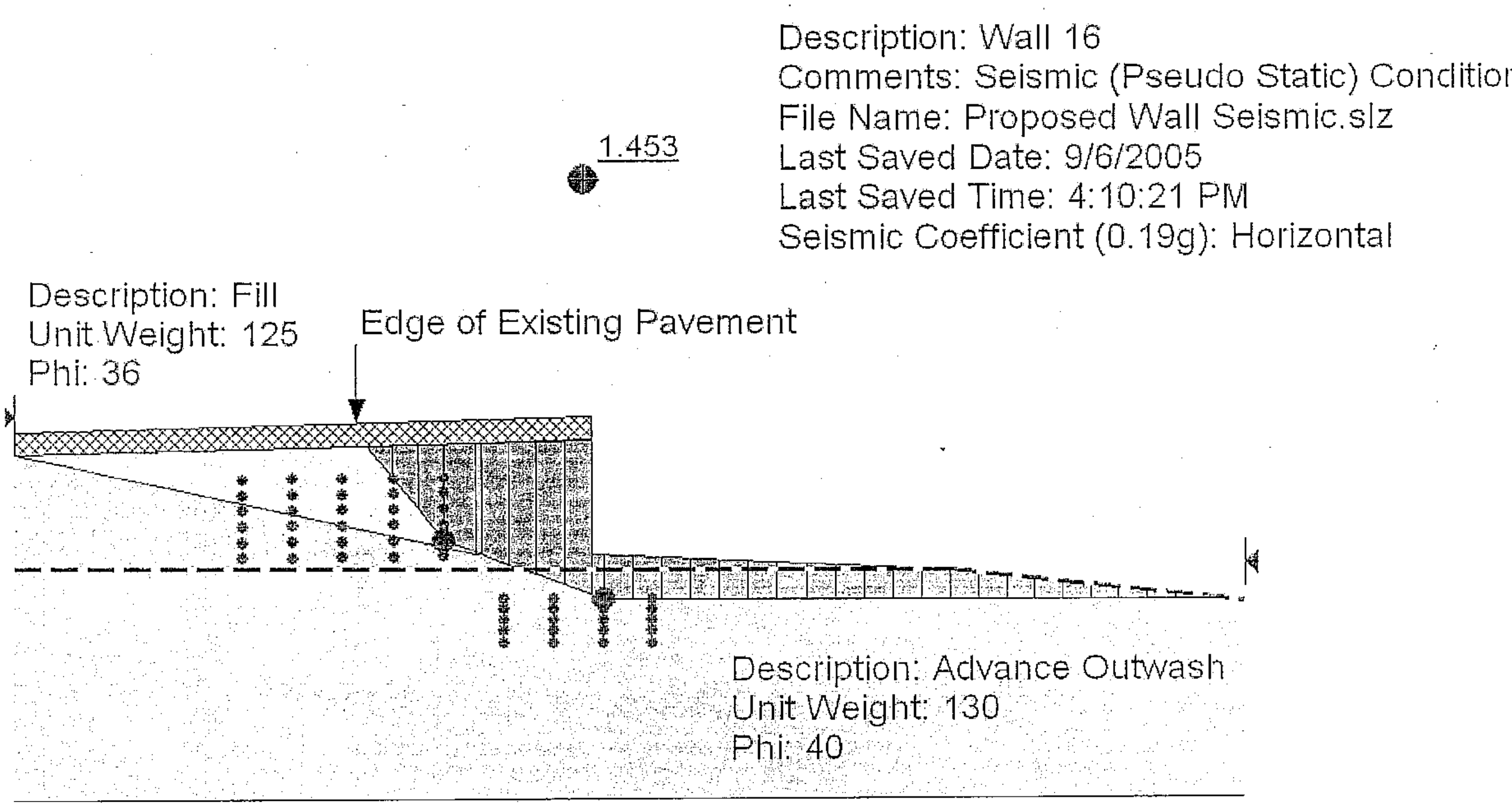


GEOENGINEERS

Wall 16 Static Condition

FIGURE C-19

0180-180-00 TB2:KGO 9-6-05 P:0\0180180\00\Working\Slope Stability Analysis\Slope Stability figures.ppt



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Wall 16 Pseudo Static Condition

FIGURE C-20

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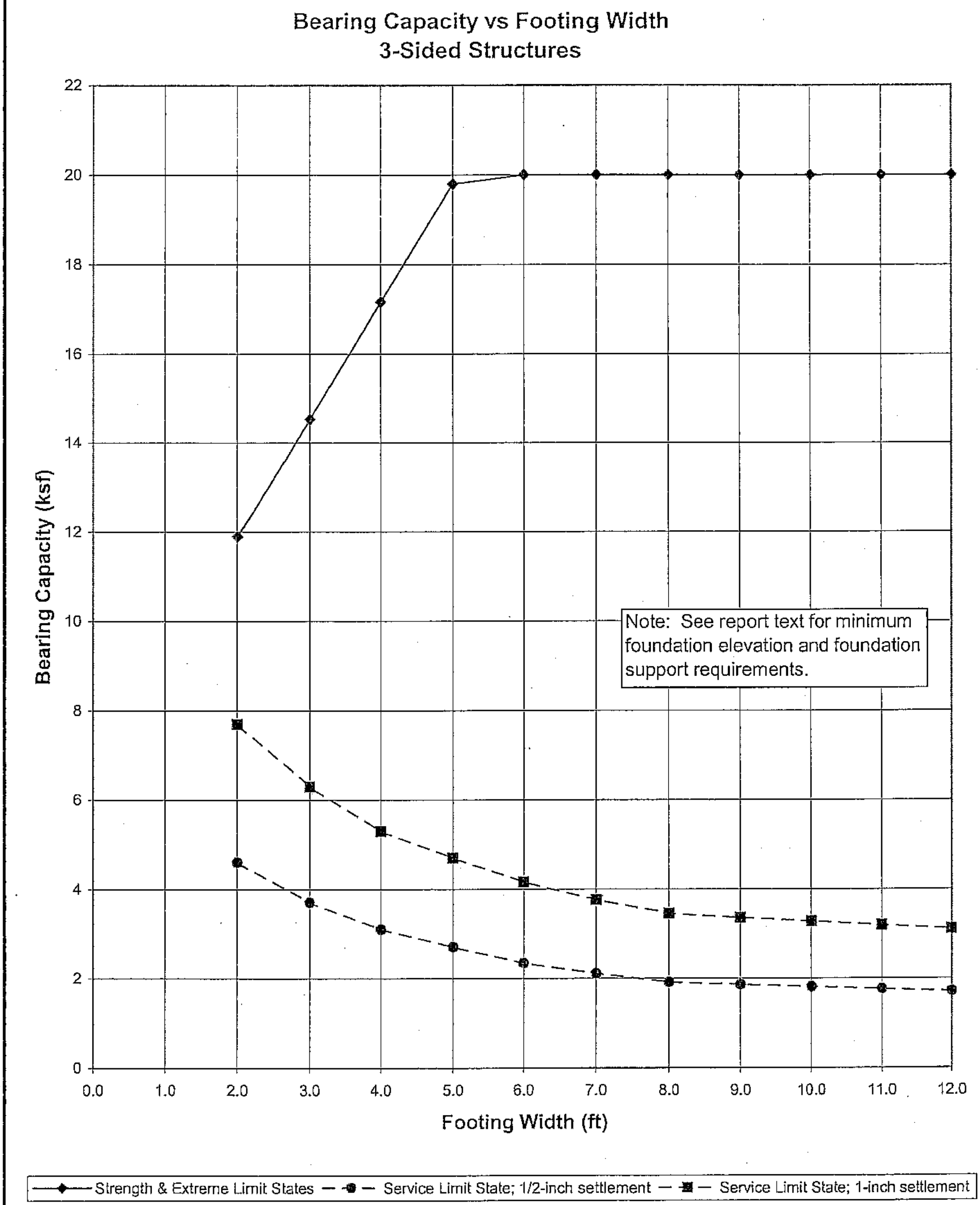
Appendix D
Shallow Foundation Bearing Capacity Charts



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Shallow Foundation Bearing Capacity Chart

Figure D-1

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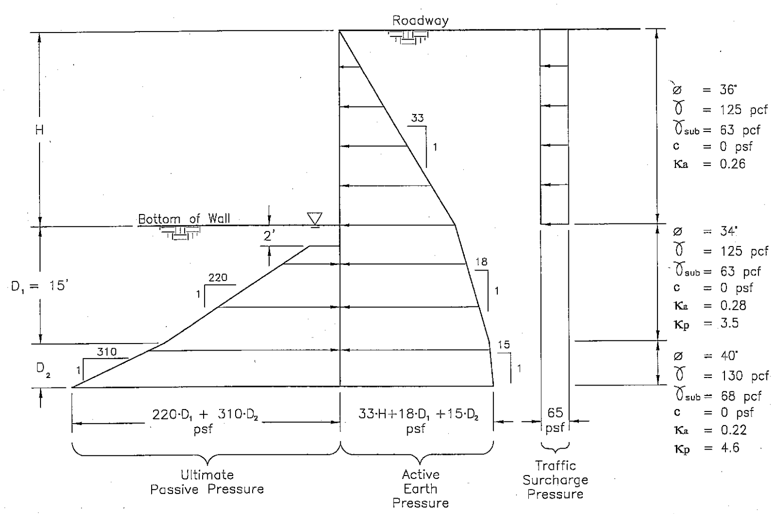
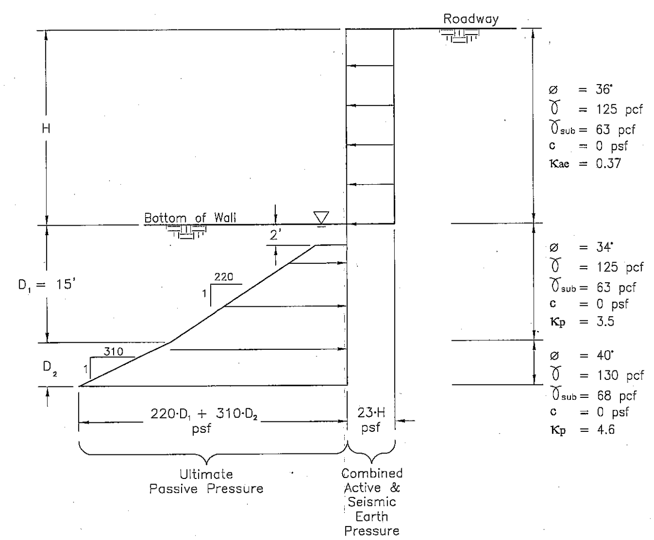
Appendix E
Soldier Pile Earth Pressure Diagram

CANTILEVER SOLDIER PILE WALL
STATIC CONDITION



NOT TO SCALE

CANTILEVER SOLDIER PILE WALL
SEISMIC CONDITION



NOT TO SCALE

Legend

H = Height of Excavation, Feet

D = Soldier Pile Embedment, Feet

Notes:
1. Active earth pressure and surcharge pressure act over the pile spacing above the base of the excavation.
2. Active earth pressure below the base of the excavation acts over one pile diameter.
3. Passive earth pressure acts over 3 times the concreted diameter of the soldier pile, or the pile spacing, whichever is less.
4. Nominal passive pressure does not include a factor of safety.
5. If additional surcharge loading (such as from soil stockpiles, excavators, dumptrucks, cranes, or concrete trucks) is anticipated, GeoEngineers should be consulted to provide revised surcharge pressures.

Earth Pressure Diagram
Cantilever Soldier Pile Wall

SR 305, OL-3420
Poulsbo SCL to Bond Road Improvements

GEOENGINEERS | **Figure E-1**

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APPENDIX F
REPORT LIMITATIONS AND GUIDELINES FOR USE

APPENDIX F
REPORT LIMITATIONS AND GUIDELINES FOR USE[1]

This appendix provides information to help you manage your risks with respect to the use of this report.

GEOTECHNICAL SERVICES ARE PERFORMED FOR SPECIFIC PURPOSES, PERSONS AND PROJECTS

This report has been prepared for the exclusive use of the WSDOT and their authorized agents. This report is not intended for use by others, and the information contained herein is not applicable to other sites.

GeoEngineers structures our services to meet the specific needs of our clients. For example, a geotechnical or geologic study conducted for a civil engineer or architect may not fulfill the needs of a construction contractor or even another civil engineer or architect that are involved in the same project. Because each geotechnical or geologic study is unique, each geotechnical engineering or geologic report is unique, prepared solely for the specific client and project site. Our report is prepared for the exclusive use of our Client. No other party may rely on the product of our services unless we agree in advance to such reliance in writing. This is to provide our firm with reasonable protection against open-ended liability claims by third parties with whom there would otherwise be no contractual limits to their actions. Within the limitations of scope, schedule and budget, our services have been executed in accordance with our Agreement with the Client and generally accepted geotechnical practices in this area at the time this report was prepared. This report should not be applied for any purpose or project except the one originally contemplated.

A GEOTECHNICAL ENGINEERING OR GEOLOGIC REPORT IS BASED ON A UNIQUE SET OF PROJECT-SPECIFIC FACTORS

This report has been prepared for the SR 305, OL-3420 Poulsbo SCL to Bond Road project. GeoEngineers considered a number of unique, project-specific factors when establishing the scope of services for this project and report. Unless GeoEngineers specifically indicates otherwise, do not rely on this report if it was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

For example, changes that can affect the applicability of this report include those that affect:

- the function of the proposed structure;
- elevation, configuration, location, orientation or weight of the proposed structure;
- composition of the design team; or
- project ownership.

If important changes are made after the date of this report, GeoEngineers should be given the opportunity to review our interpretations and recommendations and provide written modifications or confirmation, as appropriate.

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[1] Developed based on material provided by ASFE, Professional Firms Practicing in the Geosciences; www.asfe.org .

SUBSURFACE CONDITIONS CAN CHANGE

This geotechnical or geologic report is based on conditions that existed at the time the study was performed. The findings and conclusions of this report may be affected by the passage of time, by manmade events such as construction on or adjacent to the site, or by natural events such as floods, earthquakes, slope instability or groundwater fluctuations. Always contact GeoEngineers before applying a report to determine if it remains applicable.

MOST GEOTECHNICAL AND GEOLOGIC FINDINGS ARE PROFESSIONAL OPINIONS

Our interpretations of subsurface conditions are based on field observations from widely spaced sampling locations at the site. Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. GeoEngineers reviewed field and laboratory data and then applied our professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ, sometimes significantly, from those indicated in this report. Our report, conclusions and interpretations should not be construed as a warranty of the subsurface conditions.

GEOTECHNICAL ENGINEERING REPORT RECOMMENDATIONS ARE NOT FINAL

Do not over-rely on the preliminary construction recommendations included in this report. These recommendations are not final, because they were developed principally from GeoEngineers' professional judgment and opinion. GeoEngineers' recommendations can be finalized only by observing actual subsurface conditions revealed during construction. GeoEngineers cannot assume responsibility or liability for this report's recommendations if we do not perform construction observation.

Sufficient monitoring, testing and consultation by GeoEngineers should be provided during construction to confirm that the conditions encountered are consistent with those indicated by the explorations, to provide recommendations for design changes should the conditions revealed during the work differ from those anticipated, and to evaluate whether or not earthwork activities are completed in accordance with our recommendations. Retaining GeoEngineers for construction observation for this project is the most effective method of managing the risks associated with unanticipated conditions.

A GEOTECHNICAL ENGINEERING OR GEOLOGIC REPORT COULD BE SUBJECT TO MISINTERPRETATION

Misinterpretation of this report by other design team members can result in costly problems. You could lower that risk by having GeoEngineers confer with appropriate members of the design team after submitting the report. Also retain GeoEngineers to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering or geologic report. Reduce that risk by having GeoEngineers participate in pre-bid and preconstruction conferences, and by providing construction observation.

DO NOT REDRAW THE EXPLORATION LOGS

Geotechnical engineers and geologists prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering or geologic report should never be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, but recognize that separating logs from the report can elevate risk.

Give Contractors A Complete Report And Guidance

Some owners and design professionals believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering or geologic report, but preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with GeoEngineers and/or to conduct additional study to obtain the specific types of information they need or prefer. A pre-bid conference can also be valuable. Be sure contractors have sufficient time to perform additional study. Only then might an owner be in a position to give contractors the best information available, while requiring them to at least share the financial responsibilities stemming from unanticipated conditions. Further, a contingency for unanticipated conditions should be included in your project budget and schedule.

Contractors Are Responsible For Site Safety On Their Own Construction Projects

Our geotechnical recommendations are not intended to direct the contractor's procedures, methods, schedule or management of the work site. The contractor is solely responsible for job site safety and for managing construction operations to minimize risks to on-site personnel and to adjacent properties.

Read These Provisions Closely

Some clients, design professionals and contractors may not recognize that the geoscience practices (geotechnical engineering or geology) are far less exact than other engineering and natural science disciplines. This lack of understanding can create unrealistic expectations that could lead to disappointments, claims and disputes. GeoEngineers includes these explanatory "limitations" provisions in our reports to help reduce such risks. Please confer with GeoEngineers if you are unclear how these "Report Limitations and Guidelines for Use" apply to your project or site.

Geotechnical, Geologic And Environmental Reports Should Not Be Interchanged

The equipment, techniques and personnel used to perform an environmental study differ significantly from those used to perform a geotechnical or geologic study and vice versa. For that reason, a geotechnical engineering or geologic report does not usually relate any environmental findings, conclusions or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. Similarly, environmental reports are not used to address geotechnical or geologic concerns regarding a specific project.

Biological Pollutants

GeoEngineers' Scope of Work specifically excludes the investigation, detection, prevention or assessment of the presence of Biological Pollutants. Accordingly, this report does not include any interpretations, recommendations, findings, or conclusions regarding the detecting, assessing, preventing or abating of Biological Pollutants and no conclusions or inferences should be drawn regarding Biological Pollutants, as they may relate to this project. The term "Biological Pollutants" includes, but is not limited to, molds, fungi, spores, bacteria, and viruses, and/or any of their byproducts.

If Client desires these specialized services, they should be obtained from a consultant who offers services in this specialized field.